Technical Report HL-96-18
September 1996

US Army Corps
of Engineers
Waterways Experiment
Station

Channel Stability Study, Flamingo Wash, Las Vegas, Nevada

by Ronald R. Copeland, Lisa C. Hubbard

Approved For Public Release; Distribution Is Unlimited

Prepared for U.S. Army Engineer District, Los Angeles

Technical Report HL-96-18
September 1996

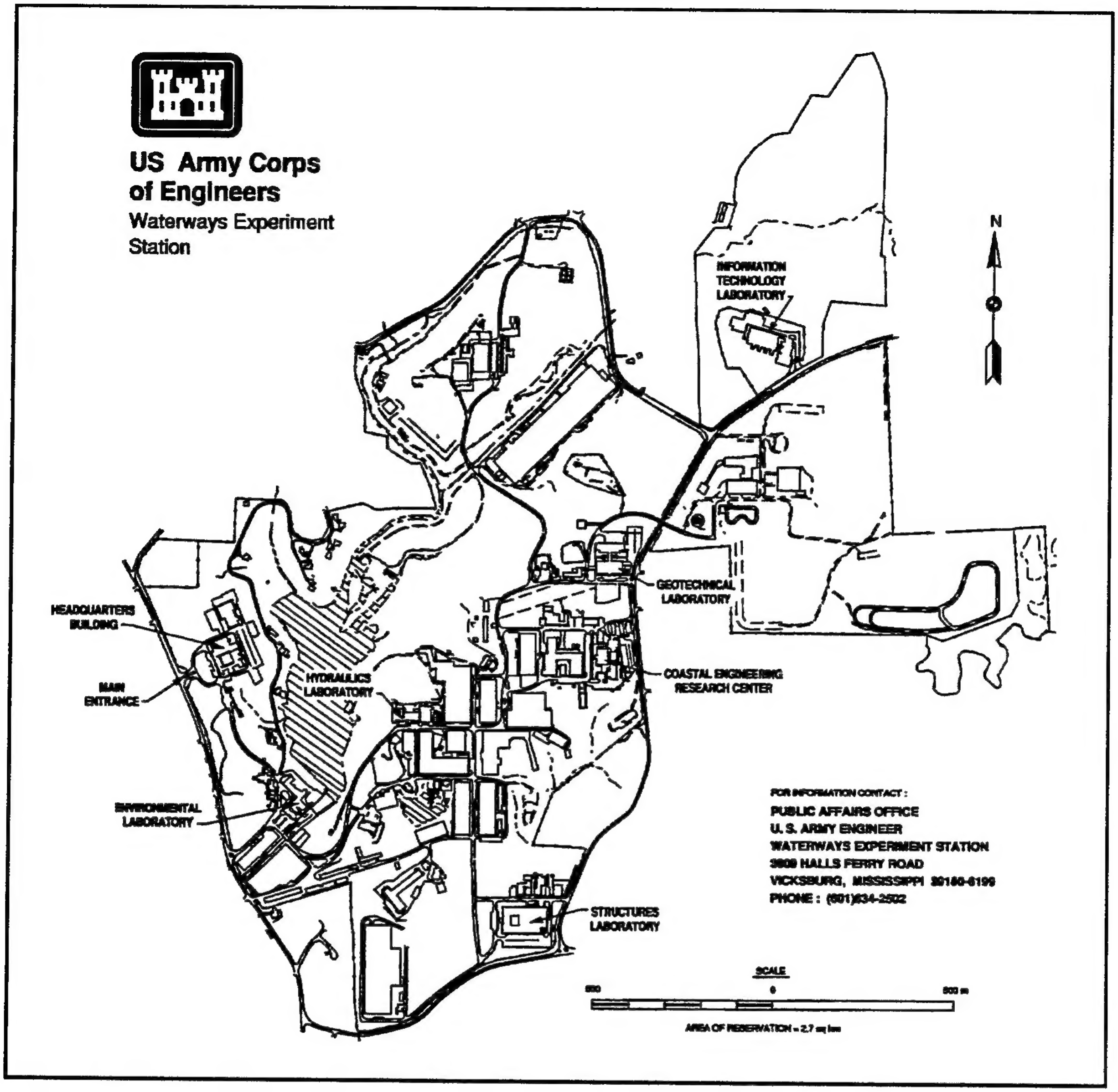
Channel Stability Study, Flamingo Wash, Las Vegas, Nevada

by Ronald R. Copeland, Lisa C. Hubbard

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

Prepared for U.S. Army Engineer District, Los Angeles
911 Wilshire Boulevard, Los Angeles, CA 90017



Waterways Experiment Station Cataloging-in-Publication Data

Copeland, Ronald R.
Channel stability study, Flamingo Wash, Las Vegas, Nevada / by Ronald R. Copeland, Lisa C. Hubbard ; prepared for U.S. Army Engineer District, Los Angeles.
99 p. : ill. ; 28 cm. — (Technical report ; HL-96-18)
Includes bibliographical references.
1. Channels (Hydraulic engineering) — Nevada — Las Vegas. 2. Flood control channels — Nevada — Las Vegas. 3. Flamingo Wash (Nev.) 4. Tropicana Wash (Nev.) I. Hubbard, Lisa C. II. United States. Army. Corps of Engineers. Los Angeles District. III. U.S. Army Engineer Waterways Experiment Station. IV. Hydraulics Laboratory (U.S. Army Engineer Waterways Experiment Station) V. Title. VI. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-96-18.
TA7 W34 no.HL-96-18

Contents

Preface

This channel stability study for Flamingo and Tropicana Washes in Las Vegas, Nevada, was conducted at the request of the U.S. Army Engineer District, Los Angeles.

This investigation was conducted during the period September 1994 to April 1996 in the Hydraulics Laboratory of the U.S. Army Engineer Waterways Experiment Station (WES), under the direction of Mr. Richard A. Sager, Acting Director of the Hydraulics Laboratory; Mr. William H. McAnally, Jr., Chief of the Waterways and Estuaries Division, Hydraulics Laboratory; and Mr. Michael J. Trawle, Chief of the Rivers and Streams Branch, Waterways and Estuaries Division. The project engineer for this study was Dr. Ronald R. Copeland, Rivers and Streams Branch. The study team included Ms. Lisa C. Hubbard, Mrs. Peggy Hoffman, and Mrs. Dinah N. McComas, Rivers and Streams Branch. Dr. Copeland and Ms. Hubbard prepared this report.

Mr. Scott E. Stonestreet served as the Hydraulic Project Engineer in the Los Angeles District and provided valuable contributions and review during the course of the study.

During the preparation and publication of this report, Dr. Robert W. Whalin was the Director of WES. COL Bruce K. Howard, EN, was the Commander.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
inches	25.4	millimeters
feet	0.3048	meters
miles (U.S. statute)	1.609347	kilometers
pounds (mass)	0.4535924	kilograms
pounds (force) per square feet	47.88026	pascals
square miles	2.589998	square kilometers

1 Introduction

Project Description

The city of Las Vegas, Nevada, is located on a desert alluvial fan which is characterized by parallel stream networks and wide shallow channels. These channels are subject to alternating erosion, deposition, and avulsion during the course of flood events, in which discharges rise and fall within hours. This natural runoff system is incompatible with the rapid urbanization which is occurring on the alluvial fan. Urban development intensifies the flood conditions by reducing rainfall infiltration and channel percolation, and by concentrating flow. To contain the additional runoff in urbanized areas, some channels have either incised or been artificially enlarged. Many channel reaches have been realigned. Although channel capacities have increased, so have velocities, depths, and channel instability. Channel instability has been addressed by numerous channel stabilization techniques including invert paving and bank protection.

The existing channels through Las Vegas Valley are not typical alluvial streams due to the abundance of caliche deposits which serve to constrain channel erosion. Caliche is a combination of gravel, sand, and desert debris which are cemented together to form a rock with varying degrees of erosion resistance. In some cases, caliche deposits form knick points or knick zones which act as local grade control and/or bank protection. In zones where caliche outcrops control the channel bed, alluvial sediment may pass over the bed as throughput. Under these conditions, the typical sediment exchange between the water column and the bed is interrupted, and classical sediment transport theory is not applicable. In some reaches, however, alluvial deposits are present, indicating that sediment transport processes are active. Based on samples collected throughout the valley (USAED, Los Angeles 1991a), the median bed-material size in these urban channels ranges between 3 and 20 mm, and the median bank material size ranges from less than 0.08 mm to 20 mm.

In recent history, the Las Vegas Valley has experienced sediment problems during several floods. Most recently, the floods of July 1975 and August 1983 were reported to have caused significant erosion and deposition problems along

the channels. The most significant problems were bank caving and deposition in existing road culverts and at bridges.

The Corps of Engineers has developed a comprehensive flood control plan for the Flamingo and Tropicana Wash drainage areas (USAED, Los Angeles 1991b). The plan includes a number of channelization projects and the construction of several temporary storage dams also referred to as dry-detention dams. These flood control reservoirs will usually be dry, but during major runoff events the dams will reduce peak discharges downstream by storing the flood runoff and releasing flow at a lower rate over a longer period of time.

The recommended flood control plan, shown in Figure 1, includes four debris basins and four detention basins. The debris basins are located at canyon mouths and are intended to capture the large sediment sizes that are carried out of steep mountain canyons. The detention basins are Red Rock, Blue Diamond, Flamingo, and Tropicana. These are designed to reduce the magnitude of peak flood discharges by providing storage and low steady releases. Red Rock and Blue Diamond detention basins are located at alluvial fan apexes and will intercept all flow and debris from the upstream watersheds. Flamingo detention basin will receive flow from Red Rock detention basin, all four debris basins, the entire Flamingo alluvial fan, and part of the Red Rock and Tropicana alluvial fans. Tropicana detention basin will receive flows from Blue Diamond and Flamingo detention basins and part of the Tropicana alluvial fan. Outflow from the entire system will be released into the existing Tropicana Wash channel, which in turn flows into Flamingo Wash and Las Vegas Wash.

Purpose of the Channel Stability Study

The Flamingo and Tropicana Wash channel stability study was conducted to assess the potential for change in channel stability associated with implementation of the flood control plan. Specifically, reaches of Flamingo and Tropicana Washes downstream from the proposed Tropicana detention basin were studied. This basin will reduce peak discharges and release flood waters at a lower discharge over a longer period of time. Approximately 1.4 miles of Tropicana Wash downstream from Koval Lane to its confluence with Flamingo Wash were studied. This reach represents the entire unpaved portion of Tropicana Wash downstream from the detention basin. Approximately 6.8 miles of Flamingo Wash between Paradise Road and Las Vegas Wash were included in the study area. A short 1.1-mile reach of Las Vegas Wash downstream from the Flamingo Wash confluence to the sewage treatment plant outfall was also studied.

The existing Tropicana and Flamingo Wash channels should be considered unstable. Channel dimensions and channel slope have not had sufficient time to

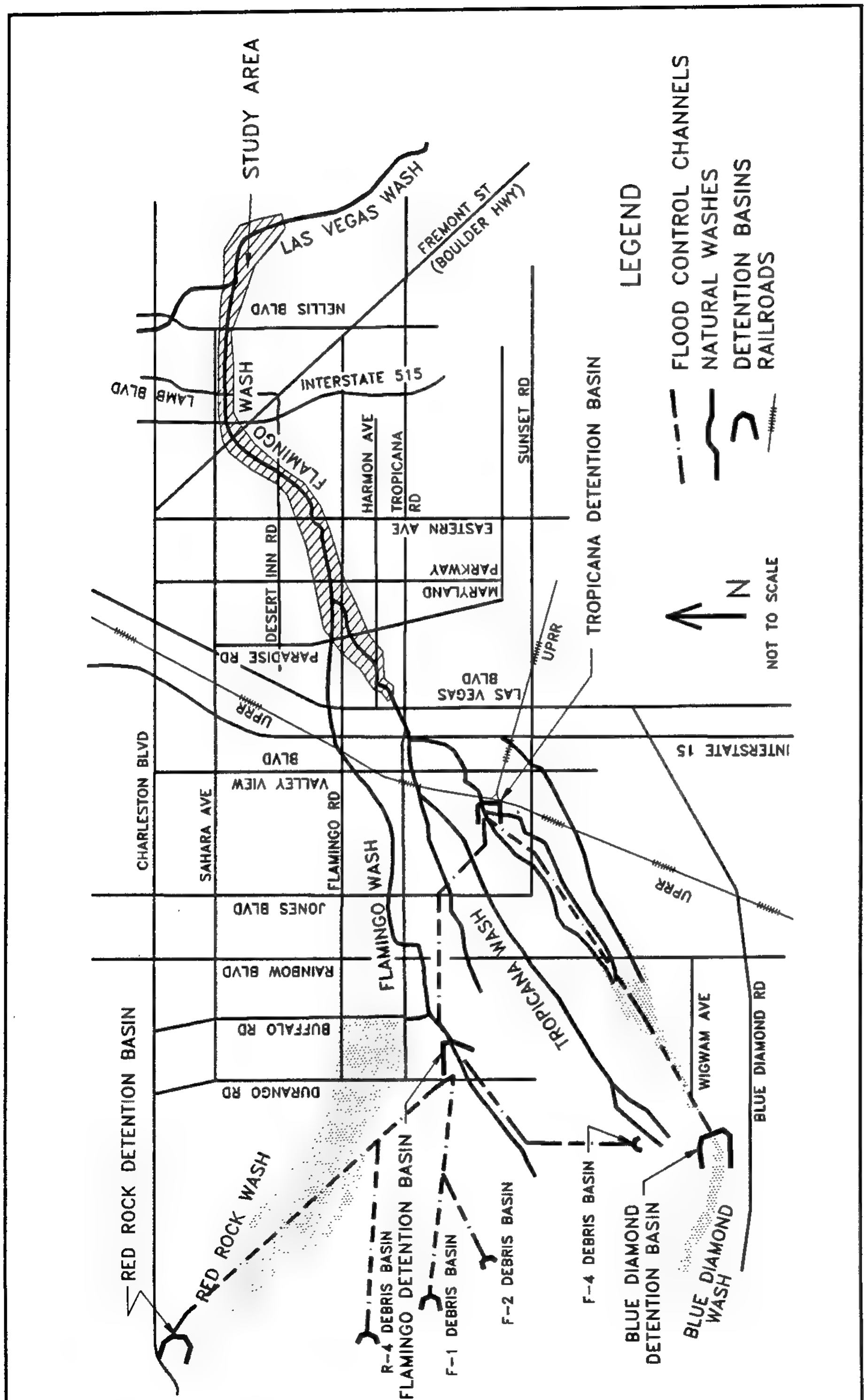


Figure 1. Flamingo and Tropicana Wash recommended Flood Control Plan

adjust to the rapidly changing hydrologic regime. The natural channel shape for these washes would be a wide shallow channel, interspersed with middle bars, rather than the existing narrow deep channel. Adequate bank protection and channel lining are required to maintain the current channel dimensions. With the existing conditions, rapid changes in channel dimensions may occur during significant flood events. These changes will occur due to bank erosion and vertical changes in the channel bed elevation.

Approach

The approach taken herein was to first identify channel reaches where stability problems were currently the most severe. This assessment was based on observations of geomorphic conditions and the proximity of structures to the channel bank. The next phase of the study was to predict vertical changes in the channel bed elevation which would occur during the course of a one-percent-chance exceedance (expected probability) flood for both existing conditions and for conditions that would exist with the flood control project. Typically, reaches with vertical channel bed instability also have high bank erosion potential. Therefore, by comparing simulated bed changes for with-project and without-project conditions, the impact of the project could be qualitatively assessed. The HEC-6W numerical model was used to calculate degradation/aggradation potential. The significant difference in project and existing conditions is the shape of the one-percent-chance exceedance hydrograph. The existing one-percent-chance hydrograph is characterized by a rapid rise and fall. The project hydrograph is also characterized by a rapid rise, but the peak is significantly less, and the falling limb of the hydrograph is characterized by a long steady discharge from the Tropicana Detention Basin. The results of this study may be used to identify reaches where the most significant amount of damage is likely during a major flood event, and to assess the relative impact of the project on channel stability.

Additionally, these results can be used by the U.S. Army Engineer District, Los Angeles, to identify reaches where the project may negatively impact channel stability such that stabilization features will be required to mitigate the impacts. These stabilization features may include bank and invert stabilization and would be included as part of the project.

2 Channel Inventory

The purpose of the channel inventory was to identify features in the existing Flamingo, Tropicana and Las Vegas Wash channels that affected channel stability. The inventory was also intended to identify the most critical locations of existing stability problems. The channel inventory included the reaches of Tropicana, Flamingo and Las Vegas Washes affected by the proposed Corps of Engineers flood control project. The Tropicana Wash reach extends about 1.4 miles downstream from Koval Lane to Tropicana Wash's confluence with Flamingo Wash. The longest channel reach inventoried was on Flamingo Wash, extending from Swenson Street for 6.8 miles downstream to Flamingo Wash's confluence with Las Vegas Wash. The inventory also included a short reach of Las Vegas Wash downstream from its confluence with Flamingo Wash for about 1.1 miles.

The study was broken into 16 reaches for descriptive purposes. These are identified in Plates 1 and 2. The initial channel inventory was based on an August 1994 field reconnaissance conducted by Dr. Ronald R. Copeland and Ms. Lisa C. Hubbard from the Waterways Experiment Station, and Mr. Scott E. Stonestreet from the Los Angeles District. This field reconnaissance and aerial photos taken in August 1993 served as the basis for the channel inventory. Subsequent field observations in April 1995, and March 1996, by Dr. Copeland and Mr. Stonestreet, revealed that significant work had been conducted in some of the channel reaches. The channel inventory was updated for this report.

Tropicana Wash
Reach 1, Between Koval Lane and Harmon Avenue

Tropicana Wash, upstream from Koval Lane, was contained in an underground culvert that extended over a mile to Interstate Highway 15. Downstream from the culvert, the channel was concrete-lined for about 150 ft through a 90 degree left angle bend. There were deflection ribs on the channel invert. Over the next 150 ft the channel had been shaped, and concrete rubble had been placed on the bank. In August 1994, the bed had an adverse slope and retained water so that the bed was heavily vegetated with grasses and

bamboo. Further downstream, over the next 200 ft, the channel was vegetated with shrubs and bushes, and the banks consisted of loose fill. There appeared to be a caliche outcrop in the bed about 250 ft downstream from the concrete-lined channel. At the mid-point between Koval Lane and Harmon Avenue, where the channel curved to the left, there appeared to be a short levee on the right bank protected by concrete rubble. An incised low-flow channel that was about 3 ft wide and 2 ft deep was observed at this location. As the channel approached Harmon Avenue, its capacity decreased significantly. The reduced capacity channel made a tight right turn and paralleled Harmon Avenue for about 200 ft. Adjacent to Harmon Avenue the channel was a grassed swale with a 2-ft-wide concrete invert. Due to the limited capacity of the grassed swale and the tight bend upstream it is expected that flood flows would leave the channel and flow eastward down Harmon Avenue. It is also expected that some of the flood breakout would be captured again on the downstream side of Harmon Avenue. A six-bay concrete-box culvert passed under Harmon Avenue. It had a total height of 8.3 ft, and in August 1994 about 1 ft of mud had deposited on the culvert invert. Plate 3 is a plan view sketch of the reach.

By March 1996 all the vegetation had been removed from the channel in this reach. Upstream from the grass swale the channel had been reshaped. The caliche outcrop in the bed 250 ft downstream from the concrete-lined channel was no longer visible. Additional portions of the bank in this reach were covered with concrete rubble. However, the channel capacity remained limited at the tight right bend upstream form Harmon Avenue.

Tropicana Wash
Reach 2, Downstream from Harmon Avenue

Downstream from Harmon Avenue, for about 150 ft, the channel was concrete-lined. At the end of the concrete-lined channel there was a 3- to 4-ft drop structure and an energy dissipator (Figure 2). Downstream from the dissipator the channel was grass-lined. Low vegetative ground cover was maintained on the side slopes between a driveway bridge, located about 250 ft downstream from Harmon Avenue, and a point 900 ft downstream from Harmon Avenue (Figure 3). Downstream from that point, both the invert and side slopes were grass-lined. Most of the grass in the channel was well maintained in August 1994, but there were a few spots where it had died and bare ground was exposed. Underneath the driveway bridge there were large cobbles and boulders that looked like they had been positioned to hold the low-flow channel. A paved dip road crossing was located about 600 ft downstream from Harmon Avenue. The grass-lined channel runs into a concrete multi-bay culvert which is located 1400 ft downstream from Harmon Avenue. The features described for reach 2 are shown in Plate 4.

Figure 2. Tropicana Wash - Reach 2, looking upstream at concrete-lined exit from Harmon Avenue culvert from driveway bridge, August 1994

Figure 3. Tropicana Wash - Reach 2, looking downstream from driveway bridge which is 250-ft downstream from Harmon Avenue, August 1994

Tropicana Wash
Reach 3, In the Vicinity of Paradise Road

This entire reach had essentially been covered and the channel contained in an 1800-ft-long culvert. The section upstream from Paradise Road was under construction in August 1994 and had been completed by April 1995. The culvert passed under the new Hard Rock Cafe Casino and Hotel. A short 20-ft-long, concrete-lined, open channel was visible just upstream from Paradise Road. The channel then entered another culvert which exited about 1000 ft downstream from Paradise Road.

Tropicana Wash
Reach 4, Between Paradise Road and Swenson Street

About 1000 ft downstream from Paradise Road, Tropicana Wash returned to an open channel. The channel had been graded to a trapezoidal shape and extended approximately 1100 ft to Swenson Street. At the upstream end of the reach, a low flow channel had been cut to a depth of about 1 ft (Figure 4).

At the upstream end of the reach, downstream from the culvert outlet, the left bank was protected by riprap for about 50 ft. The left side slope was graded for the next 270 ft and was composed of loose natural material. Gabions, filled with caliche, had been placed at the toe of the left bank for about 120 of the 270-ft-long section. For the next 200 ft downstream, the left bank appeared to rest on a natural caliche outcrop. The left bank was covered by broken caliche riprap for the next 460 ft. The right bank, downstream from the culvert outlet, consisted of graded loose natural material. Upstream from Swenson Street, for 340 ft, both the right and left channel banks consisted of vertical concrete walls which rested on an exposed caliche outcrop. Above the vertical concrete walls the channel side slopes were covered with gravel. The right bank had broken concrete riprap for an additional 100 ft upstream. The channel entered a six-bay, 4-ft-high box culvert at Swenson Street. Sand and mud covered the invert of the left five bays in August 1994. To help in the location of these features refer to Plate 5.

In August 1994, caliche outcrops were observed in the bed of the channel throughout this reach. Significant outcrops spanning the width of the channel were observed just upstream from Swenson Street and about 200 ft downstream from the outlet at the upstream end of the reach. Saturated caliche deposits in this reach were found to be softer than dry deposits. Fine sand had deposited in the invert, probably from the graded banks and bed. Channel vegetation consisted of grass and weeds which had turned yellow, possibly due to a herbicide treatment.

Tropicana Wash
Reach 5, Between Swenson Street and Flamingo Road

This reach consisted of a 600-ft-long graded trapezoidal channel. Natural caliche outcrops were present in the bed, and hard deposits were observed in the banks. In August 1994, grasses and scattered small bushes were present in the channel, but some of the vegetation had turned yellow, possibly due herbicide treatment. The vegetation was healthy in April 1995.

Layered deposits were observed in a three-ft-high vertical cut just upstream from Flamingo Road. As shown in Figures 5 and 6, the lower layer, which was composed of sand and pebbles, was being undercut and had a small detritus deposit at the toe. The sand and pebble layer was covered by a series of layers composed of finer material, another sand and pebble layer, and then additional layers of finer material. This suggests that bank erosion potential along the channel may vary considerably depending on the characteristics of the alluvial deposits that make up the bank at specific locations. These characteristics are typically hidden by channel grading projects and are visible only when erosion exposes the actual bank composition. Considering that the banks in this reach were observed to have layers composed of non-cohesive sand and pebbles, bank erosion potential should be considered high. Materials composing these layers are more easily removed by flowing water and are more subject to blowout by positive pore pressure in the bank than cohesive materials.

There was a major caliche drop located about 50 ft upstream from Flamingo Road. The total vertical drop was about four feet (Figure 7). The caliche drop demonstrated both the potential for degradation in this reach and the ability of the caliche deposits to form a bed control. Broken caliche fragments were observed downstream from the drop which demonstrated that the deposit was subject to failure and could not be considered a permanent hydraulic control structure. Upstream from this large drop, the caliche outcrops were not visible across the full channel width.

At Flamingo Road the channel entered a triple concrete-box culvert. Sloping pier noses should decrease the potential for floating debris build-up at the culvert entrance. Sediment deposits were observed in the culvert during the August 1994 field reconnaissance. About 6 to 12 inches of broken caliche and mud had deposited in the left bay. At the culvert exit, downstream from Flamingo Road, the natural channel bed was about 1 ft lower than the concrete invert. Plate 6 is a plan view sketch of the features in this reach.

Figure 4. Tropicana Wash - Reach 4, looking downstream from culvert outlet toward Swenson Street, August 1994

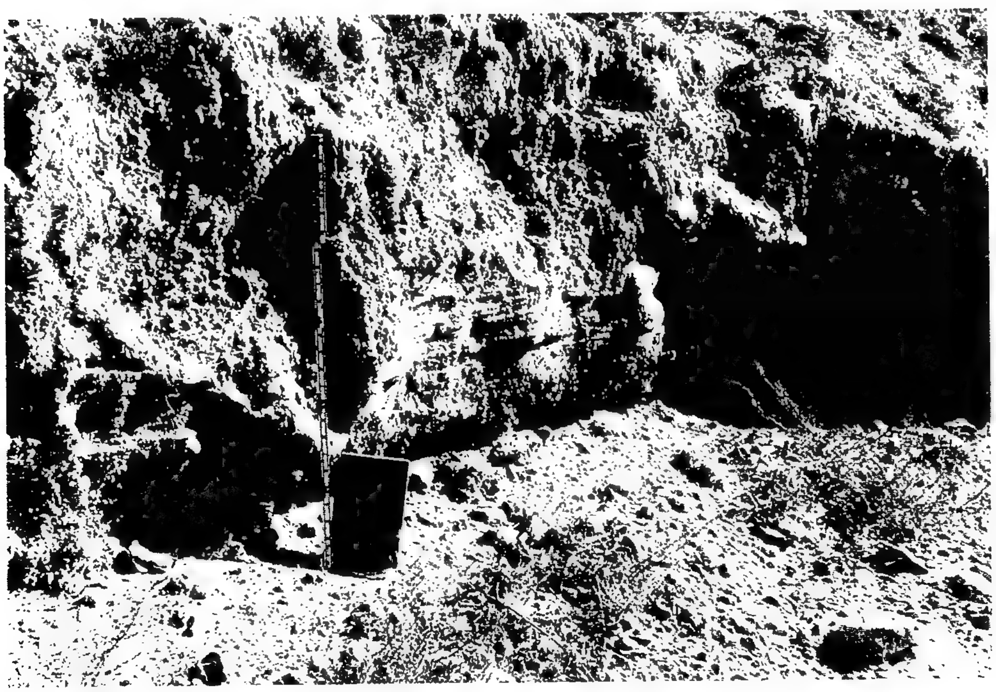


Figure 5. Tropicana Wash - Reach 5, vertical cut in right bank just upstream from Flamingo Road, August 1994

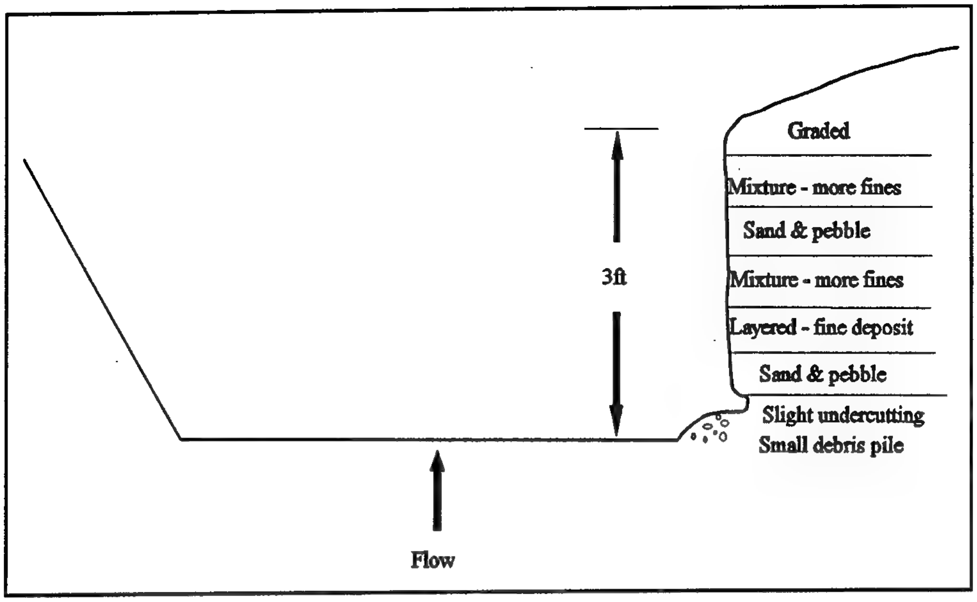


Figure 6. Tropicana Wash - Reach 5, layer description of vertical cut in right bank just upstream from Flamingo Road

Tropicana Wash and Flamingo Wash Reach 6, Confluence Area

The 1993 aerial photographs showed that this reach contained significant vegetation, including mature trees, brush and grasses. However, by August 1994, the area had been cleared of all vegetation, and the channels had been graded into generally trapezoidal shapes on both Tropicana and Flamingo Washes, with 4- to 5-ft-deep low-flow channels of variable width. The Tropicana Wash channel downstream from Flamingo Road is shown in Figure 8, and the Flamingo Wash channel upstream from the confluence with Tropicana Wash is shown in Figure 9.

At the upstream end of reach 6 on Flamingo Wash was the Swenson Street culvert. The bays of the triple concrete-box culvert were 7.5 by 10 ft. In April 1995, the left and right bays had deposits of broken caliche and gravel. The culvert had sloping pier noses. Flamingo Wash upstream of Swenson Street had extensive caliche deposits in the bed.

Bank protection measures were minimal in reach 6. Rock and soil-filled gabions were located on the upper left bank of Tropicana Wash, just downstream from Flamingo Road. These would be ineffective when exposed to sustained flow because much of the material inside the wire baskets was smaller than the wire mesh openings. Scattered concrete and caliche rubble had been placed on the upper left bank of Tropicana Wash downstream from Flamingo Road all the way to the adjoining right bank of Flamingo Wash. The gabions and rubble would have limited effectiveness in preventing bank erosion

because the toe elevations were located on a soft berm above the low-flow channel.

Caliche deposits were abundant on both Tropicana and Flamingo Washes in reach 6. Some of the caliche appeared to be solid, but at other locations it was difficult to determine the extent of the deposit because channel reshaping work had left accumulations of broken caliche in the channels. Both the solid caliche deposits and the broken caliche accumulations will function as channel bed controls. There were two vee-shaped caliche controls observed on Tropicana Wash. The lower one had a four-ft drop and consisted of broken caliche deposits, while the upper one had a drop of about 2 ft through a solid caliche deposit. On Flamingo Wash there was 1.5 ft caliche drop through a solid deposit just upstream from the confluence and a 4-ft drop that had cut about a 25-ft-long trench through a solid caliche deposit located further upstream. From the confluence to Cambridge Street, downstream, was a solid caliche outcrop that extended all the way across the channel invert. A plan view sketch of this reach is shown in Plate 7.

At Cambridge Street there was a triple concrete-box culvert. In August 1994, caliche, sand, and mud had deposited to depths of one to two ft in the culvert.

Flamingo Wash
Reach 7, Between Cambridge Street and Maryland Parkway

Downstream from Cambridge Street the graded channel generally had a trapezoidal shape with a 10-ft-wide low-flow channel cut to a depth of 4 to 5 ft (Figure 10). The nearly vertical upper banks consisted of sand and gravel. The bank material was quite hard even though the presence of significant cohesive material was not apparent. The upper layers of these banks were composed of loose material. Concrete-lined bank protection extended about 100 ft upstream from Maryland Parkway on the right bank, and about 300 ft upstream on the left bank. The concrete-lined side slopes were not keyed into the bed. Generally, the bottom elevation of the slab was at the existing channel bed elevation. However, erosion had occurred at the base of the lined side slopes at several locations (Figure 11). Further erosion could undercut the slabs and cause failure along a significant reach of the channel. Geometric irregularity associated with slab failure would accelerate the local erosion process. A plan view of reach 7 is shown in Plate 7.

The culvert at Maryland Parkway was a four-bay concrete-box culvert. There was a fully-lined concrete sluice upstream and a 2.5-ft vertical drop and concrete apron downstream. The total drop through the structure was about 6 ft. Scour at the end of the apron was about 2 ft in August 1994 but had been backfilled to only a few inches by April 1995.

Figure 7. Tropicana Wash - Reach 5, looking upstream from Flamingo Road at caliche bed control, August 1994

Figure 8. Tropicana Wash - Reach 6, looking downstream from Flamingo Road, April 1995

Figure 9. Flamingo Wash - Reach 6, looking upstream from the Tropicana Wash confluence, April 1995

Figure 10. Flamingo Wash - Reach 7, looking downstream from Cambridge Street, April 1995

Flamingo Wash
Reach 8, Between Maryland Parkway and Spencer Street

The first 1400 ft of channel downstream from Maryland Parkway was a shaped channel with a 10-ft-wide low-flow channel which was about 4 ft deep. Vegetation was sparse in this reach with a few scattered bushes. The bed slope just downstream from Maryland Parkway was noticeably steeper than further downstream. Caliche was observed in the bed for the first few hundred feet downstream from Maryland Parkway. The upper right banks of the channel were nearly vertical with a layer of loose material on top. The vertical banks showed layering of sand and gravel, sand, and mixtures, as shown in Figures 12 and 13. These banks were quite hard but did not appear to have significant cohesive material present. Rills, created by parking-lot runoff, were observed at several locations along the right bank. This overbank erosion will weaken the bank. Small rock had been dumped into one of the rills. On the left bank, a gabion retaining wall that extended for 700 ft downstream from the wing walls at Maryland Parkway had been constructed between August 1994 and April 1995. Downstream from the gabions, concrete rubble had been dumped at the toe on the left bank for an additional 300 ft. A plan view sketch of this reach is shown in Plate 8.

About 1400 ft downstream from Maryland Parkway, the channel was fully concrete-lined for about 800 ft (Figure 14). Tennis courts had been constructed over the channel about midway along the concrete-lined channel. On the upstream side of the tennis courts there was a 2.5-ft drop and downstream, another 2.5-ft drop. In August 1994, a 5-ft-deep scour hole was observed at the end of the concrete-lined channel (Figure 15). Foundation material from under the concrete slab had been scoured. In addition, immediately downstream from the end of the concrete-lined channel the scour was five feet below the toe of concrete-lined slide slopes on the left bank and had undercut the slab. By March 1996, angular gravel-size material had been imported to this site filling the scour hole and returning the bed elevation adjacent to the side slopes to the base elevation of the concrete-lined side slope.

Downstream, the remaining 800-ft of reach 8 had a trapezoidal channel with concrete-lined side slopes and a natural invert (Figure 16). The natural invert had sparse grass on the bed, and a low flow channel had developed to a depth of about 1 ft in August 1994. Degradation of about 2 to 3 ft had occurred below the toe of the concrete side slopes at some locations. No evidence of concrete footings was observed. By March 1996, berms had been constructed from imported angular gravel-sized material along both banks throughout this reach. Just upstream of Spencer Street there was a concrete access road that faced upstream on the right bank. This would act as a launch for flood flows.

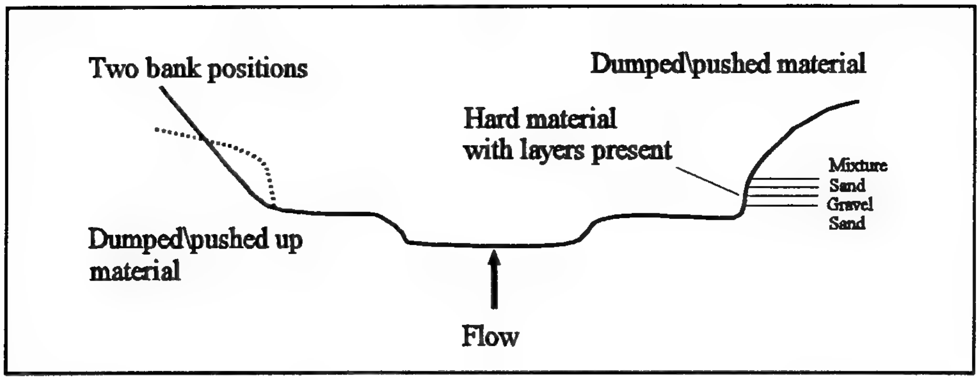


Figure 13. Flamingo Wash - Reach 8, Typical channel cross section between Maryland Parkway and concrete-lined channel

The four-bay concrete-box culvert at Spencer Street was 5 ft high and had sloping pier noses. There was a 6-ft-high sloping drop downstream from the culvert. A plan view sketch of this reach is shown in Plate 9.

Flamingo Wash
Reach 9, Between Spencer Street and Eastern Avenue

Downstream from the Spencer Street culvert was a short transitional channel leading into a more natural channel through a golf course. The first 150 ft downstream from the culvert had concrete-lined side slopes and an unprotected invert. There was a 2.5-ft-high sill at the end of the concrete-lined side slopes. Downstream from the sill the banks were composed of loose graded material. The channel bed through the golf course appeared to be unmaintained with significant vegetation. Gravel bars were observed in the bed. The mild sloping banks were covered with well maintained grass (Figure 17).

At Tioga Way the channel entered three corrugated-metal-pipe arch culverts. The channel upstream and downstream was vegetated with reeds and cat tails, and the banks were covered with well maintained grass. Just upstream of Tioga Way was a clear span golf cart bridge with evidence of pipe and wire revetments along the channel banks. Downstream of Tioga Way were two clear span golf cart bridges.

The channel-banks were concrete lined upstream from the Eastern Avenue culvert for about 100 ft. The concrete-box culvert had five bays and was 7 ft high. There was a grouted stone apron and grouted stone wingwalls downstream from Eastern Avenue. About 2 ft of degradation was observed downstream from the grouted stone apron.

Figure 14. Flamingo Wash - Reach 8, looking downstream at entrance to concrete-lined channel, August 1994

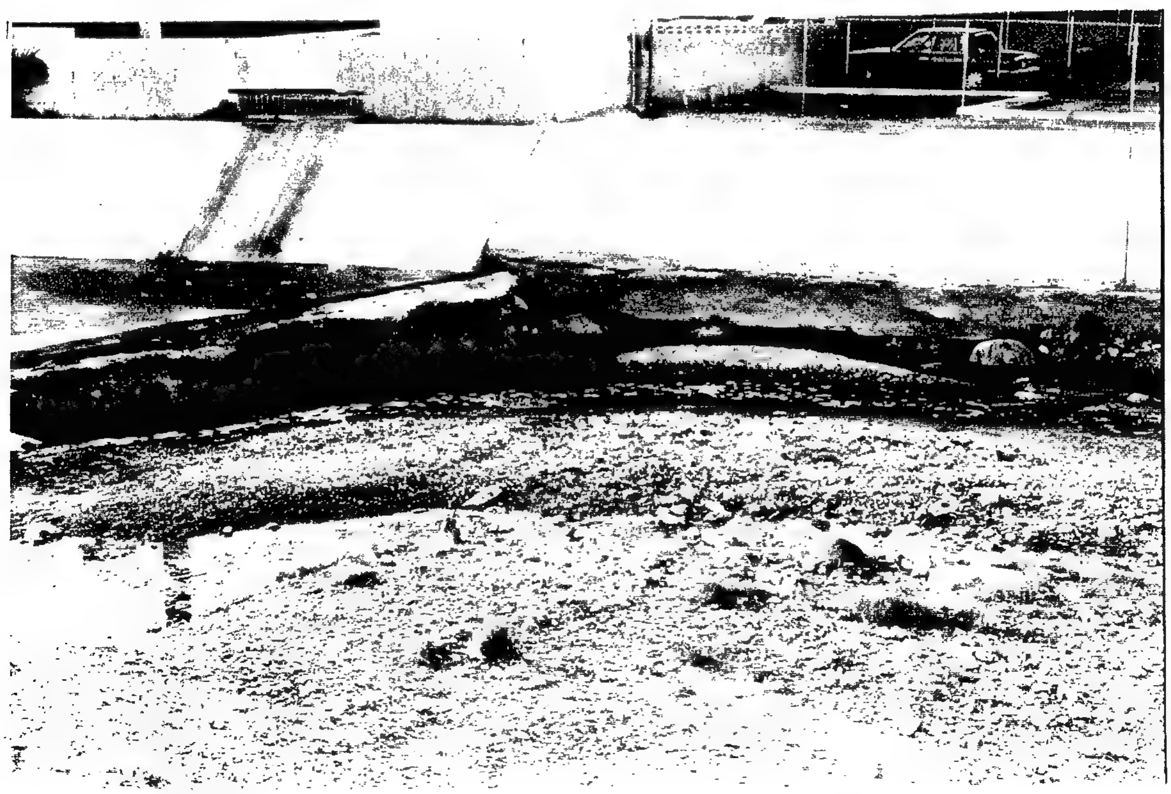


Figure 15. Flamingo Wash - Reach 8, erosion at end of concrete-lined channel, April 1995

Figure 16. Flamingo Wash - Reach 8, looking downstream toward Spencer Street, April 1995

Figure 17. Flamingo Wash - Reach 9, looking downstream through golf course, near Spencer Street, April 1995

Flamingo Wash
Reach 10, Between Eastern Avenue and McLeod Drive

Downstream from Eastern Avenue the right bank was protected by concrete-lined side slopes for about 200 ft. One of the concrete slabs was observed to be launching. This failure appeared to be caused by local drainage behind the slab. In August 1994, an active erosion scar was observed on the left bank. The bank was composed of loose material. A graded trapezoidal channel extended for about 300 ft downstream from Eastern Avenue. Some vegetation was observed in the channel. About 100 ft downstream from Eastern Avenue were two caliche outcrops in the low flow thalweg, each with about a 1-ft drop. Within the next 100 ft was another caliche outcrop with about a 1-ft drop. A pipe crossing was observed about 250 ft downstream from Eastern Avenue that also had caused about a 1-ft drop in the low flow thalweg.

In August 1994, the channel became more densely vegetated in a downstream direction after the first 300 ft downstream from Eastern Avenue. Bushes and trees in the channel were as high or higher than the channel banks. This reach was heavily vegetated all the way to McLeod Drive, making access difficult. The channel banks in this reach consisted of loose material that appeared to have been dumped or pushed up. At some locations the banks had been covered with irregular dumped rubble or broken concrete blocks.

About 900 to 600 ft upstream from McLeod Drive a short canopied reach was observed which had a 10-ft-wide gravel bed and tree-covered banks (Figure 18). The right bank consisted of dumped material. A caliche outcrop was observed in the bed.

About 300 ft upstream from McLeod Drive was a major caliche outcrop with a 10-ft drop. The left bank appeared to consist of dumped material, but a gravel lens was observed. The density of vegetation decreased downstream from the outcrop. The right bank consisted of caliche at the base with layered sand and pebble material on top. This material was fairly hard but undercutting was observed in the softer layers underneath it. This outcrop had forced the flow towards the left bank which was made of softer material, and erosion was occurring.

By April 1995 significant changes had occurred in this reach. Complete channel clearing and reshaping had taken place. The erosion scar on the left bank just downstream from Eastern Avenue had been regraded, as had the entire cross-section. Broken caliche and rubble had been dumped along the bank at several locations along the reach. The low flow channel dimensions varied but were generally 10 ft wide and 1 to 2 ft deep, becoming 2 to 3 ft deep closer to McLeod Drive. The bed material varied from gravel to fine sand. There was no evidence of caliche outcrops except for the 10-ft drop upstream of McLeod Drive, which had been broken somewhat by construction

equipment. However, broken caliche fragments were present in the bed at several locations along the reach. The bank material was very loose. This reach has a very high potential for erosion and instability (Figure 19).

Between Eastern Avenue and McLeod Road were traces of an older incised channel that had meandered within a much wider irregular bankline. The old channel remnant had been subject to land filling.

The bridge under McLeod Drive had concrete-lined side slopes and a natural invert. There was a gravel bar on the left bank, and undercutting of the slab on the right abutment was observed in August 1994. Evidence that repair work had occurred on the right abutment was observed. A plan view sketch of this reach is shown in Plate 10.

Flamingo Wash
Reach 11, Between McLeod Drive and Desert Inn Road

The channel between McLeod Drive and Desert Inn Road consisted of a shaped trapezoidal channel with a left bend for 600 ft, followed by a right bend for 600 ft (Plate 11). A low-flow channel had been cut to a width of about 15 ft and a depth of 1-2 ft. The depth of the low flow channel increased to 4 ft as the channel approached Desert Inn Road. Only sparse grass was observed in the channel. In August 1994, the low-flow channel had grass and reeds in the bed near Desert Inn Road. Continuous concrete rubble bank protection had been placed on the outside banks. The remainder of the banks had very sparse concrete rubble. Desert Inn Road had a five bay concrete box culvert with sloping pier noses.

Flamingo Wash
Reach 12, Between Desert Inn Road and Mojave Road

This reach was a fully-lined concrete channel. In August 1994, sparse sediment deposits were observed on the channel invert. Some grasses and reeds had established themselves on the sediment deposits. Under Mojave Road was a five-bay concrete-box culvert with a 2.5-ft vertical drop to the waterline downstream. In addition to the 2.5-ft drop, a 4-ft scour hole was observed downstream from the sill, which included 16 in of scour under the slab itself.

Figure 18. Flamingo Wash - Reach 10, canopied channel just upstream from McLeod Drive, August 1994

Figure 19. Flamingo Wash - Reach 10, looking downstream, about halfway between Eastern Avenue and McLeod Drive, April 1995

Flamingo Wash
Reach 13, Between Mojave Road and Vegas Valley Drive

Downstream from Mojave Road was a shaped trapezoidal channel with a low-flow channel about 8 ft deep by 20 ft wide (Figure 20). Riprap had been placed on the right outer bank of the low-flow channel. A mobile armor layer was present in the bed at some locations. A caliche control with a 1.5-ft drop was located about 1300 ft downstream from Mojave Road near a local drainage inlet. A point bar had developed downstream from the caliche outcrop. Bank erosion was observed in the low-flow channel in areas without riprap protection. Between August 1994 and April 1995 the longitudinal extent of the riprap protection in the low-flow channel had been increased significantly. Bank erosion observed at several locations in August 1994 had been repaired. In unprotected reaches, there was a natural meander pattern present in the low-flow channel that had a much smaller wave length than the main channel curvature. This resulted in bank erosion on the inside of the channel bed as shown in Figure 21. The composition of the low-flow channel banks in this reach was varied. There was sand in the lower layers, and either weakly cemented sand and gravel or just loose sand and gravel in the upper layers. The bank angles observed were relatively steep, which was surprising considering the non-cohesive appearance of the bank material. The banks seemed natural but asphalt and trash were present. The banks may have been shaped using dumped material and then reworked by the stream, resulting in the layered appearance.

Downstream, towards Vegas Valley Drive, the low-flow channel widened and became shallower. The bed seemed to consist of a gravel mobile armor layer, and the bank material was unconsolidated. The notes in Plate 12 point out the variability of the low flow channel in this reach.

Flamingo Wash
Reach 14, Between Vegas Valley Drive and Boulder Highway

Downstream from Vegas Valley Drive the channel flowed through the Miracle Mile and Kings Row trailer parks. The channel was narrower than upstream. In August 1994, the channel was heavily vegetated with high grasses, reeds, heavy brush and trees. Gravel bars were observed on the bed of the channel, which appeared to consist of material finer than upstream. The banks were composed of fill material and covered with scattered concrete rubble. There was some erosion of the banks behind the vegetation. Recent high flows had laid down some of the vegetation in the channel. The footing

under the trailer park bridge was being undercut on the right bank. The bed under the bridge was natural.

By March 1996, all the vegetation had been removed from this reach of Flamingo Wash, exposing the natural banks to erosion.

There were two concrete-box culverts under Boulder Highway, which is a divided highway.

Flamingo Wash Reach 15, Between Boulder Highway and Interstate 515

In August 1994 there was a concrete-lined apron downstream from Boulder Highway that ended with a 2.5-ft drop over a jagged end indicating a failed sill due to downstream degradation. Both banks were protected by dumped rubble for about 600 ft. Grouted stone riprap protected the left bank for an additional 300 ft. The bed had mobile gravel armor, and some vegetation was present. Towards Interstate 515 the banks became rugged and less well defined. The abutments of the Interstate 515 bridge were protected with gabions. The bridge itself had concrete side slopes and a natural invert that showed signs of degradation. This whole reach was in the process of being reshaped in March 1996. A sketch plan map for this reach is shown in Plate 13.

Flamingo Wash Reach 16, Between Interstate 515 and Lamb Boulevard

Downstream from Interstate 515 was a graded trapezoidal channel with a cut low-flow channel. The low-flow channel became more poorly defined as it approached Lamb Boulevard. The main channel in this reach was wider, and the banks were not as steep as further upstream. In August 1994 meandering was observed in the 1- to 2-ft deep and 30- to 40-ft wide low-flow channel. Gabions placed on the right bank, just downstream from Interstate 515, were in danger of failing due to backwashing. Other than this short reach of gabions there was no bank protection in the reach. In August 1994 grass that had been growing in the channel bed was yellow and appeared to have been recently sprayed with herbicide. The low-flow channel had been reshaped by April 1995, removing signs of deposition and meandering (Figures 22 and 23).

The channel is fully concrete-lined for about 150 ft upstream from Lamb Boulevard. There was a nine-bay concrete-box culvert under Lamb Boulevard.

Figure 20. Flamingo Wash - Reach 13, looking downstream from Mojave Road, April 1995

Figure 21. Flamingo Wash - Reach 13, downstream from Mojave Road looking at erosion on Inside of channel bend, April 1995

Figure 22. Flamingo Wash - Reach 16, looking upstream from Lamb Boulevard, August 1994

Figure 23. Flamingo Wash - Reach 16, looking upstream from Lamb Boulevard, April 1995

Downstream was a two-stage concrete drop structure. The first drop was 3 ft 7 inches, and the second was 3 ft 2 inches.

Flamingo Wash
Reach 17, Between Lamb Boulevard and Nellis Boulevard

There were three sub-reaches between Lamb Boulevard and Nellis Boulevard. Significant changes occurred in these sub-reaches between August 1994 and April 1995. The first sub-reach started downstream from Lamb Boulevard and extended 1200 ft. In August 1994 the channel had concrete-lined side slopes for 150 ft downstream from the concrete drop structure. Downstream the banks were steep and irregular, and an incised channel had been cut. There were erosion scars on the right bank. There was a natural caliche head cut with a 4-ft drop about 30 ft downstream from the lined side slopes. This reach was steep with a total drop of about 5-6 ft (Figure 24). By April 1995 the channel had been regraded into a trapezoidal shape, removing all evidence of erosion or degradation (Figure 25).

In August 1994 there was a large hard caliche outcrop about 1000 ft downstream from Lamb Street. Within this outcrop a 6-ft headcut was highly visible (Figure 26). There was an additional 1- to 2-ft drop upstream from the headcut. The material in the caliche outcrop itself appeared to be hard, but it was underlain with sand. The flow was slowly cutting through the hard layer, rapidly eroding the underlying sand when it became exposed. Degradation of the big waterfall was already visible at its weaker portions, suggesting that over a long time period it would not be stable.

By April 1995 this subreach had been reshaped, and most evidence of the head cut was gone. Vegetation had been cleared from the reach. A 15-ft-wide and 3-ft-deep low-flow channel had been established. About 600 ft downstream from Lamb Boulevard a weir had been constructed. The weir had a concrete sill and a gabion apron. Gabions also extended up the side slopes. The weir consisted of two 16-inch drops. A remnant of the caliche drop, 1000 ft downstream from Lamb Boulevard, was still visible (Figure 27). A second gabion weir was located 1400 ft downstream from Lamb Boulevard and had the same drop as the first (Figure 28).

The second sub-reach started about 1400 ft downstream from Lamb Boulevard and extended 1600 ft. In this sub-reach there was a vertical natural bank on the left in August 1994. Grouted stone bank protection had been placed on the upper bank without any toe protection for a distance of 300 ft. Whether the grouted stone had been poured this way or degradation had cut beneath the grouted stone was not apparent by observation. Either way the grouted stone would have eventually failed. The grouted stone bank protection had been repaired by April 1995. The bank was no longer being undercut, and

additional toe protection had been added. It appeared that material had been imported to create a berm adjacent to the low-flow channel. On the right bank about 600 ft from the start of the sub-reach, there was a short reach of grouted stone for about 150 ft. Undercutting of this bank protection seemed likely in August 1994, but reshaping of the channel by April 1995 erased any signs of potential erosion problems.

The final sub-reach started 3000 ft downstream from Lamb Boulevard and extended 2100 ft to Nellis Boulevard. The channel slope became milder in this subreach, encouraging aggradation. The trapezoidal channel in this sub-reach had no bank protection. Some vertical banks were observed in August 1994. These banks were composed of fairly hard sandy material that was not cohesive. Large sections of the right bank had failed recently and showed evidence of having been regraded. The low-flow channel was meandering straight into this right bank and had caused an erosion problem (Figure 29). The bed had a mobile gravel armor. Some portions of the bed were composed of fine sand and silts. In April 1995 the banks had been reshaped, the low-flow channel redirected, and there was no evidence of meandering or bank erosion.

A concrete-box culvert carries the flow under Nellis Boulevard. There is a two ft drop at the culvert exit.

Flamingo Wash
Reach 18, Nellis Boulevard to Las Vegas Wash

Downstream from Nellis Boulevard the channel had concrete-lined side slopes and a natural invert for about 50-100 ft. The channel then flowed into a golf course for the next 4200 ft where the channel consisted of a grass swale. The grass was dry in August 1994. Low flows were diverted into an underground culvert that returns to Flamingo Wash just before it joins Las Vegas Wash, about 4500 ft downstream from Nellis Boulevard.

Las Vegas Wash
Reach 19, Flamingo Wash to Vegas Valley Drive

The reach started at the end of the golf course about 5000 ft downstream from Nellis Boulevard and continued for about 1650 ft to Vegas Valley Drive. Las Vegas Wash upstream from its confluence with Flamingo Wash was also a grass-lined swale through a golf course. Las Vegas Wash downstream of the confluence had been shaped into a trapezoidal channel with some concrete blocks present for bank protection. Some shrubs had become established in the bed. About 450 ft downstream from the start of this reach there was a sheet pile stabilizer.

Figure 24. Flamingo Wash - Reach 17, looking downstream from Lamb Boulevard, August 1994

Figure 25. Flamingo Wash - Reach 17, looking downstream from Lamb Boulevard, April 1995

Figure 26. Flamingo Wash - Reach 17, caliche headcut, August 1994

Figure 27. Flamingo Wash - Reach 17, caliche headcut, April 1995

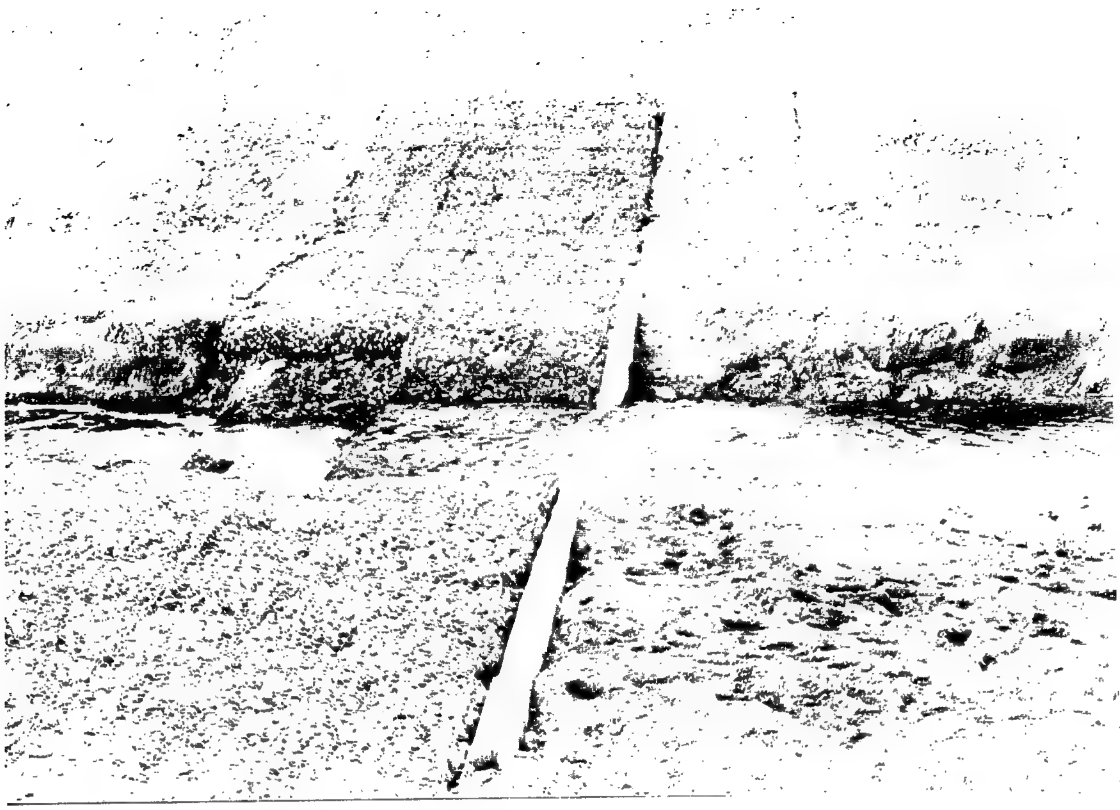


Figure 28. Flamingo Wash - Reach 17, gabion grade control in low flow channel, April 1995

Figure 29. Flamingo Wash - Reach 17, erosion on right bank, upstream from Nellis Boulevard, August 1994

The channel was concrete-lined for the next 1800 ft, all the way to Vegas Valley Drive. Range Wash joined Las Vegas Wash at the upstream end of this lined channel and was also fully concrete lined at the confluence.

Las Vegas Wash
Reach 20, Vegas Valley Drive to the Sewage Plant Outflow

The channel was still fully concrete-lined under Vegas Valley Drive for about 450 ft downstream. After this point the channel became a shaped trapezoidal channel with rubble and broken concrete blocks on the banks. The natural channel bed was vegetated. No major head cuts were observed downstream of the concrete-lined channel or downstream of the sewage outflow pipe, but vegetation obscured some of the channel. Further field reconnaissance is suggested in this reach.

Summary

Degradation in Flamingo and Tropicana Washes is checked by numerous bed controls present in the system. The locations of both man-made and natural bed controls are shown in Plates 14 and 15.

Man-made controls include pipe crossings, concrete-lined inverts, sills, and stabilizers. Generally, the concrete invert controls would be areas of long term stability for the bed. Scour typically occurs downstream from concrete inverts and was observed downstream of almost every concrete culvert and channel in the study reach. Scour holes greater in depth than downstream cutoff walls could undermine the structure causing failure. Downstream cutoff walls are therefore essential features of concrete invert designs. Examples of concrete inverts being undercut in August 1994 included the concrete channel in reach 8, the culvert at Mojave Road in reach 13, and the downstream apron at Boulder Highway in reach 16.

Numerous caliche outcrops serve as natural bed controls. The fact that caliche bed controls are present suggests that the channel has degradation potential and that incision has been halted or slowed by the presence of the caliche. In the long term, caliche is still subject to erosion and is not as stable as reinforced concrete. The outcrops themselves vary in hardness and therefore have different resistivity to erosion. When the caliche was wet, both hardness and resistance were reduced. The extent of the outcrop is important when evaluating stability. To have the maximum stabilizing effect the outcrop has to extend over the full width of the channel. Broken caliche blocks, as opposed to solid outcrops, are less resistant. For a more detailed mapping of these caliche outcrops and their relative hardness further field reconnaissance

would be required. Reaches 4, 5, and 6 on Tropicana and Flamingo Washes had significant caliche deposits. Major caliche bed controls, associated with significant channel drops, were observed in Tropicana Wash in reach 5 upstream from Flamingo Road, Flamingo Wash reach 6 upstream from its confluence with Tropicana Wash, Flamingo Wash reach 10 upstream from McLeod Drive, and Flamingo Wash reach 17 downstream from Lamb Boulevard.

Channel types were classified based on observations made during the August 1994 field reconnaissance and are shown on Plates 16 and 17. The fully concrete-lined reaches may be considered stable except at the downstream ends where there may be undercutting. However, channels with concrete-lined banks that have inadequate toe protection are not considered stable. For example, refer to reach 6 just upstream of Maryland Parkway and reach 8 between the concrete-lined channel and Spencer Street.

Channel vegetation will affect both channel erodibility and channel conveyance. Grass-lined channels provide erosion resistance without adversely affecting the conveyance. Examples are reach 2 and reach 18. Grass, scattered brush, and/or cat tails in shaped channels slightly reduce both channel conveyance and channel erodibility. When the channel is heavily vegetated with brush and trees channel conveyance can be drastically reduced. As of March 1996 there were no remaining reaches with significant conveyance-reducing vegetation. However, with time, and/or lack of maintenance, vegetation could re-establish itself and once more reduce conveyance.

By March 1996, most of the non-concrete-lined channel reaches studied could be classified as graded trapezoidal channels with high bank erosion potential. Most of these channel reaches had cut low-flow channels. The banks of these channels consisted of loose material and would be very unstable during flood events and very prone to erosion. Most of the reaches have some rubble or concrete blocks present on the banks or along the low-flow channel that would provide some protection against erosion. The reliability of this protection under various flows is uncertain.

3 Numerical Model

Description

The HEC-6W one-dimensional numerical sedimentation program was used to develop the numerical model for this study. Development of this computer program was initiated by Mr. William A. Thomas at the U.S. Army Engineer District, Little Rock, in 1967. Further development at the U.S. Army Engineer Hydrologic Engineering Center (USAEHEC) and at the U.S. Army Engineer Waterways Experiment Station (WES) by Mr. Thomas has produced the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (USAEHEC 1993). Additional modifications and enhancements to the program by Mr. Thomas and others at WES led to the HEC-6W program currently in use. This study was conducted using version 4.00, dated March 1996. The HEC-6W code applied to Flamingo Wash uses a different armoring algorithm and a slightly different backwater calculation than the 1993 version of HEC-6. The program produces a one-dimensional model that simulates the response of the riverbed profile to sediment inflow, bed-material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events and their effects on the sediment transport capacity at cross sections and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method. The program assigns critical depth for water-surface elevation if the backwater calculations indicate transitions to supercritical flow. However, for supercritical flow, hydraulic parameters for sediment transport are calculated assuming normal depth in the channel.

For numerical sedimentation models to completely simulate the behavior of a stream channel, computations would have to account for all of the basic processes of sedimentation: erosion, entrainment, transportation, deposition, and compaction of both the bed and streambanks for the complete range of particle sizes found in nature. The state of the art has not advanced to such a complete simulation. The computer program used in this study, HEC-6W, is a state-of-the-art program for use in mobile-bed channels. It incorporates procedures for describing the complex sedimentation processes when these procedures have been established by research and published. Where knowledge gaps exist, the HEC-6W program contains logic that bridges those

gaps. When applied by experts using good engineering judgement, the HEC-6W program will provide good insight into the behavior of mobile-bed channels. Because the program has given reliable results at similar projects, it is expected to give reliable answers to questions being addressed here.

The reaches of Flamingo and Tropicana Washes studied herein are atypical of alluvial streams in that the bed is not fully mobile. Significant lengths of the streambed consist of caliche outcrops which have variable degrees of resistance to erosion. These resistant formations may be observed at the streambed surface, they may exist just below the surface, or they may not be present at all. A detailed geologic survey would be required to determine the exact extent of the caliche outcrops. This was beyond the scope of this study, which was intended to determine the **potential** for vertical channel instability, i.e. degradation and/or aggradation. The numerical model assumes a fully mobile bed, although a fixed bed may be assigned to represent outcrops where no degradation is possible. Due to the lack of knowledge relative to the depth of the available bed sediment reservoir and the uncertainty related to sediment inflow and bank erosion, results of the numerical model simulations are not appropriate for determining exact quantities of streambed change. However, the results are useful for comparing the potential for vertical channel instability for existing conditions and for postproject conditions.

Numerical Model Geometry

The geometry for the numerical model was based on aerial photography taken in September 1993, which produced 2-ft-contour interval topographic maps. Cross-sections were digitized from the digital terrain model developed from the aerial photos by the U.S. Army Engineer District, Los Angeles, and supplied to WES. Subsequent to the development of the aerial photos, significant channel improvement work was accomplished by the local flood control authority. These improvements were not incorporated into the model geometry. However, changes in channel roughness due to channel improvements and removal of vegetation were incorporated into the numerical model. The major channel changes listed in the following tabulation were observed during field trips after September 1993. Minor reshaping of channels, especially low-flow channels, was observed in many locations.

The downstream boundary of the numerical model was on Las Vegas Wash at station 100+00. The model extended up Las Vegas Wash, about 2.8 miles, to station 248+00. Flamingo Wash was modeled from its confluence with Las Vegas Wash for about 6.8 miles to Paradise Road. Tropicana Wash was modeled from its confluence with Flamingo Wash to Koval Lane, about 1.4 miles.

	Stations	Change
Flamingo Wash	46+00 - 96+00	Channel reshaped - 2 gabion drops installed
Flamingo Wash	140+00 - 162+00	Vegetation removed
Flamingo Wash	214+00 - 217+00	Channel reshaped
Flamingo Wash	217+00 - 242+00	Vegetation removed and channel reshaped
Flamingo Wash	278+00 - 286+00	Backfill with riprap on bed
Flamingo Wash	301+00 - 308+00	Gabion bank protection on left bank
Flamingo Wash	323+00 - 334+00	Vegetation removed and channel reshaped
Tropicana Wash	0+00 - 7+00	Vegetation removed and channel reshaped
Tropicana Wash	38+00 - 45+00	Channel replaced by culvert
Tropicana Wash	63+00 - 73+00	Vegetation removed and channel reshaped

Hydrology

One-percent chance exceedance hydrographs were determined by the Los Angeles District for existing conditions (USAED, Los Angeles, 1991c) and for with-project conditions as defined in the Feasibility Report (USAED, Los Angeles, 1993). The peak discharges were based on expected probability. The hydrographs were developed synthetically with the storms centered to achieve one-percent-chance exceedance peaks for both with and without project conditions. Storm centerings were therefore different for the two conditions. The comparison herein therefore is related to the hydrograph frequency differences and not uniquely to the effect of the detention basin itself. The hydrographs were developed only for Tropicana and Flamingo Washes. No coincident contribution from Las Vegas Wash was included.

The most significant difference between the existing and project conditions in the numerical model simulations was the hydrographs. The duration of the without-project hydrograph is just over one day. The with-project hydrograph has a duration of about 5.5 days. The one-percent chance exceedance hydrographs for Flamingo Wash at its confluence with Las Vegas Wash are compared in Figure 30. A schematic of hydrology input, with peak discharges, is shown in Figure 31. Also shown in Figure 31 are numerical model control points and locations of local inflows. The negligible 10-cfs input on Las Vegas Wash was included to achieve numerical stability in the model.

Computational time steps in the numerical model varied between 1 and 15 minutes.

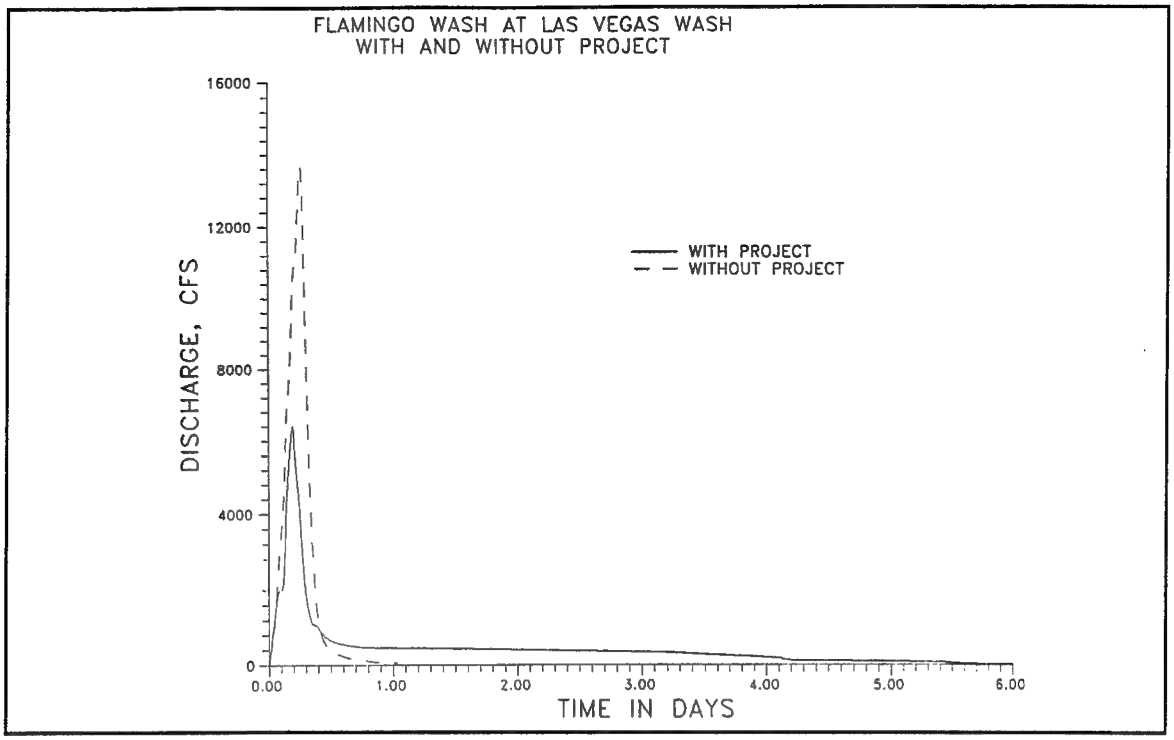


Figure 30. One-percent-chance exceedance hydrographs for Flamingo Wash at Boulder Highway

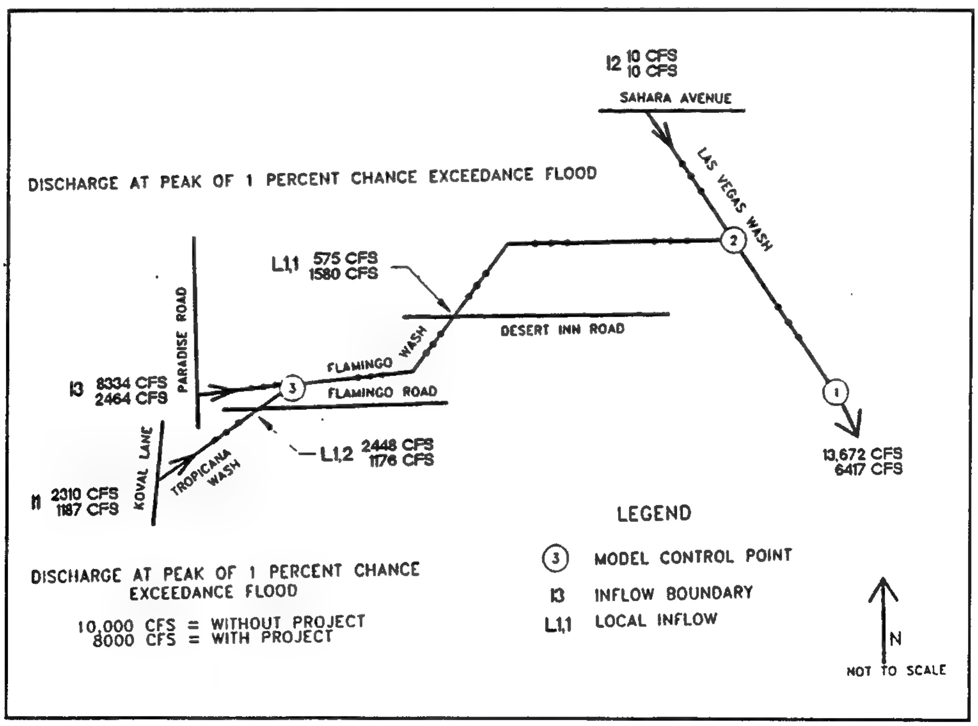


Figure 31. Hydrology input schematic, for HEC-6W numerical model

Bed Sediment Reservoir

At cross-sections upstream from each bridge and culvert, where there was a concrete invert, the bed in the numerical model was fixed so that no bed sediment was available. This included every street crossing except McLeod Drive and Interstate 515. Many bridges and culverts had significant drop structures immediately downstream. These were simulated in the numerical model by a cross-section downstream from the drop and the fixed-bed cross-section upstream from the bridge or culvert. In addition, drop structures and concrete-lined channels were assigned fixed beds. These included drop structures on Tropicana Wash downstream from Harmon Avenue, concrete-lined channels in Flamingo Wash between Maryland Parkway and Spencer Street and between Desert Inn Road and Mojave Road, a sheet-pile stabilizer on Las Vegas Wash downstream from its confluence with Flamingo Wash, and a concrete lined channel on Las Vegas Wash downstream from its confluence with Flamingo Wash.

The presence of caliche beneath the channel bed made it difficult to determine the potential for channel degradation and the availability of bed sediment to supply the sediment transport deficit. The channel inventory identified several caliche outcrops that were of sufficient magnitude that they could be considered a permanent grade control point. These were assigned fixed beds in the numerical model. Other caliche outcrops were identified but the extent of the formation was uncertain. These were assigned a bed sediment reservoir depth of two feet, allowing for some contribution to the sediment supply. This limits the calculated depth of scour at these sections to two feet. However, the actual potential for scour may be much greater. The initial bed sediment reservoir depth was set at ten feet at all other locations. However, during the course or the study, the bed sediment reservoir depth was reduced to five feet at two locations to prevent excessive calculated scour and contribution to the downstream sediment load.

The initial bed-material gradation used in the numerical model was based on six sediment samples taken along Tropicana and Flamingo Washes (USAED, Los Angeles 1991a). The range and median gradation are shown in Figure 32. Einstein (1950) recommended that the finest ten percent of sampled bed material should be excluded from bed-material load calculations. He reasoned that this fine material is most likely a remnant of deposition at low flow of fine material trapped in the coarse surface-layer matrix. In this study, size classes between 0.25 mm and 120 mm were included in the model when the Meyer-Peter Muller (1948) sediment transport equation was used. Size classes between 0.5 and 120 mm were used with the Laursen-Copeland function (Copeland and Thomas 1989).

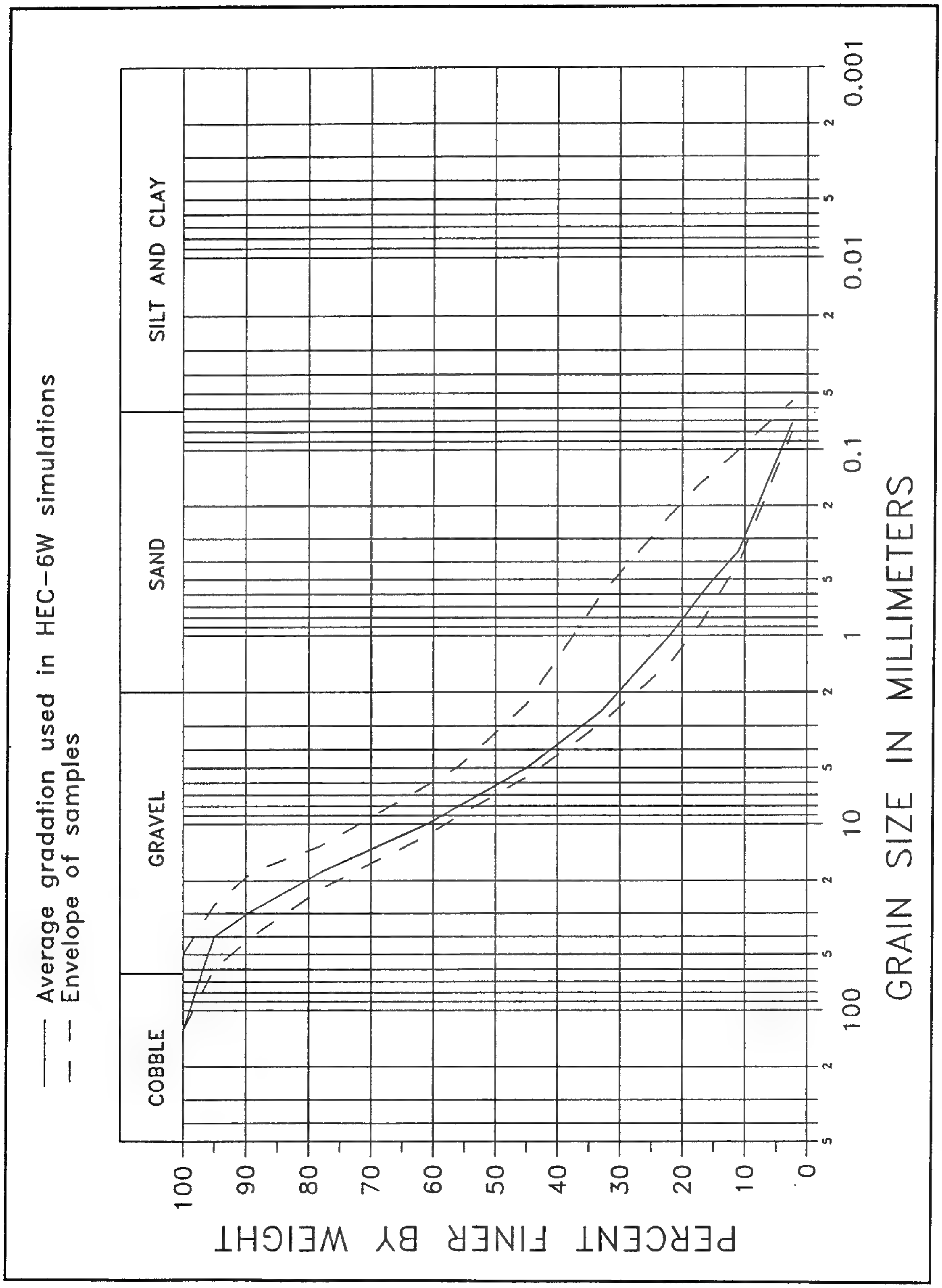


Figure 32. Bed-material gradations - Tropicana and Flamingo Washes

Sediment Inflow

Although the quantities are unknown, sediment inflow to Flamingo and Tropicana Washes will be reduced by the project. Measured sediment concentration data were available for neither Tropicana nor Flamingo Washes. In addition, there were no measured aggradation or degradation data for numerical model adjustment or circumstantiation.

Sediment inflow at the upstream end of the model was calculated using the Hydraulic Design Package SAM (Copeland, McComas, Raphelt and Thomas 1996). The supply reach for Tropicana Wash was taken to be the channel reach upstream of Interstate 15 between Industrial Avenue and Valley View Road. An HEC-2 backwater model was developed using cross-section data taken from an HEC-6 numerical model developed by the Los Angeles District in 1991 (USAED Los Angeles 1991a). Cross-sections for that model were based on 4-ft-contour-interval topography, possibly enhanced by field surveys, and were used in the FIS study by J.M. Montgomery Consultants (1988). The supply reach for Flamingo Wash was taken to be the reach upstream from Interstate 15 between the UPRR and Valley View Road. A representative cross-section was determined for this reach using the same four-ft-contour-interval topography. The same bed-material gradation used in the HEC-6W model was used to calculate sediment transport. Sediment-inflow rating curves were calculated for the full range of discharges in the one-percent chance exceedance hydrograph.

It was apparent from field observations that not all of the sediment transport load calculated for the supply reaches would reach the upstream numerical model boundary. The supply reaches were not in equilibrium. Both supply reaches showed signs of degradation, and significant portions of the channel bed in both reaches had bedrock, caliche, or immobile particles on the surface. This suggests that assuming the entire bed to be available for supply and transport of sediment is unrealistic. The Tropicana Wash channel disappears as it crosses Industrial Avenue, so its sediment load will tend to deposit before reaching the Interstate 15 culvert. In addition, high flows will pond behind the Interstate 15 culverts on both washes, reducing sediment transport capacity. On Flamingo Wash, high flows will be diverted away from the channel along the UPRR tracks, reducing sediment transport capacity downstream. Both washes must traverse tortuous paths through culverts beneath Las Vegas strip casinos and hotels, especially Flamingo Wash which winds its way through an underground parking lot beneath the Imperial Palace Hotel and Casino. It was concluded that reducing the calculated sediment inflow by 50 percent would provide reasonable upstream boundary conditions for the numerical model.

On Flamingo Wash, the same sediment inflow rating curve was specified for the with-project and without-project simulations. Although the sediment rating curve was the same, the volume of sediment entering Flamingo Wash was much less for with-project conditions because only local drainage contributed to flood flows.

On Tropicana Wash, the calculated sediment inflow rating curve, which had been reduced by 50 percent, was used for the without-project simulation. Sediment inflow was specified as zero for the with-project conditions. A concrete-lined channel is planned between the Tropicana detention basin and the Interstate 15 culvert.

Sediment-Transport Functions

The Laursen-Copeland sediment transport function was chosen to calculate aggradation and degradation in the numerical model. This function was developed for streams that transport both sand and gravel size classes. The Meyer-Peter Muller sediment transport function was also used to determine how sensitive the calculated results were to the sediment transport function. The Meyer-Peter and Muller equation was developed for bed load calculations in gravel-bed streams.

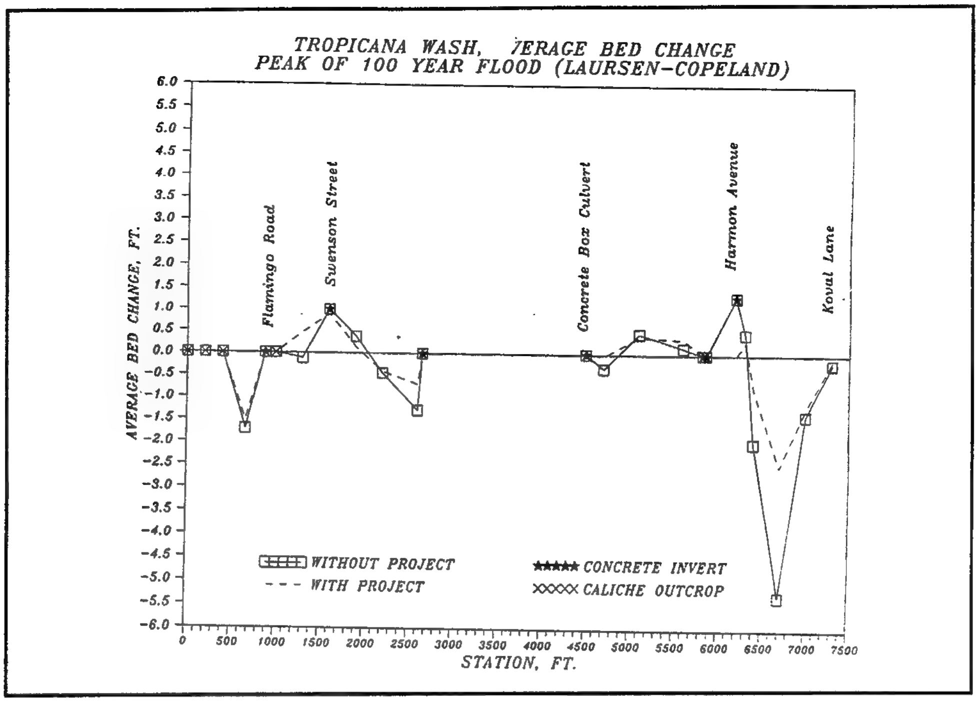
A combination of the Toffaleti (1968) and Meyer-Peter and Muller sediment transport functions was used in a previous study (USAED, Los Angeles 1991a) and was the initial choice for this study. The Toffaleti function was developed for sand-bed rivers. The two sediment transport functions have been combined in HEC-6W and SAM to provide a sediment transport equation for use in streams that transport both sand and gravel as bed load and suspended load. However, it was determined during the course of this study that the Toffaleti function was very sensitive to the calculated bed-material gradation during the solution of the sediment continuity equation. In reaches with fixed beds, i.e. concrete-lined channels, calculated transport using the Toffaleti equation was unrealistically low. As a result aggradation in the concrete-lined channels was predicted. This result was deemed unreasonable. The problem was overcome by using the Laursen-Copeland function.

4 Numerical Model Results

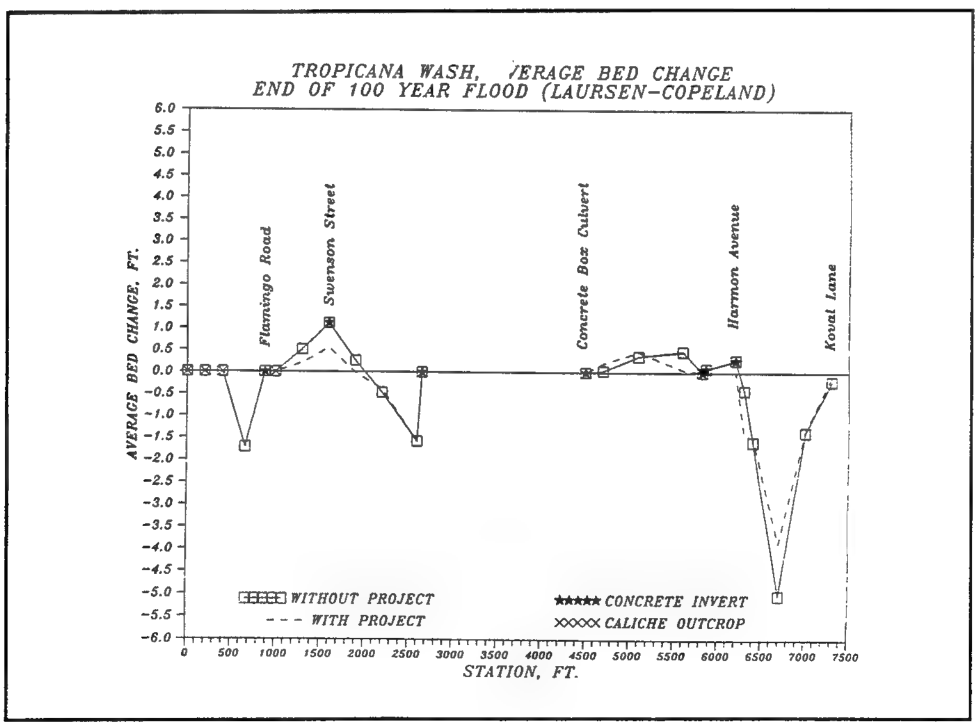
General

Average bed elevation changes were calculated for the with-project one-percent-chance exceedance hydrograph and the without-project one-percent-chance exceedance hydrograph. Conditions at the peak of hydrograph are shown in Figures 33a-37a; and conditions at the end of the hydrograph are shown in Figures 33b-37b. Results are also tabulated in Tables 1 and 2. In most reaches, most of the calculated degradation occurred on the rising limb of the hydrograph, so that there was typically little additional degradation after the peak discharge. Typically, zones of degradation occurred downstream from culverts and concrete channel reaches. Aggradation frequently occurred downstream from degradation reaches or just upstream from culverts. In general, aggradation and degradation are significantly less severe for the with-project condition. It can be inferred that decreasing the aggradation and degradation potential will also decrease channel instability.

Outflow from the proposed dam will discharge into Tropicana Wash just downstream from Koval Lane. Due to upstream diversions, the total volume of water flowing through Tropicana Wash will increase with the project, even though the peak discharges will be less. This increase in flow volume has the potential for increasing degradation. In addition, the project will reduce sediment supply to Tropicana Wash, which could further increase the potential for degradation. This reach is therefore a relatively sensitive reach. Significant degradation was calculated on Tropicana Wash downstream from Koval Lane for both with- and without-project conditions and both at the peak and at the end of the flood hydograph. Interestingly, calculated erosion was less for the with-project condition, even though sediment inflow into Tropicana Wash was eliminated by the project. At the peak of the flood hydrograph, deposition was calculated on Tropicana Wash at Harmon Avenue where the channel size is significantly reduced. Downstream from Harmon Avenue, through the grass-lined reach 2, channel bed elevation changes are relatively small on Tropicana Wash. Some degradation occurs in the channel as it exits the culvert downstream from Paradise Road, but the degradation is limited by caliche outcrops. Degradation potential is high on Tropicana Wash downstream from Flamingo Road, but caliche outcrops will tend to hold the

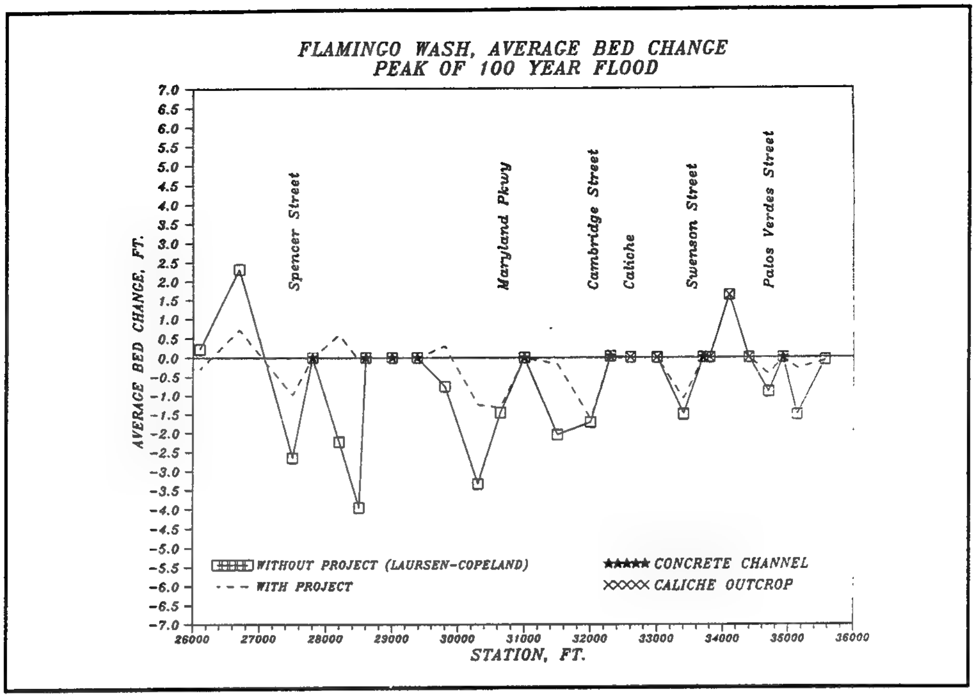


a. Peak of the 1-percent-chance exceedance hydrograph

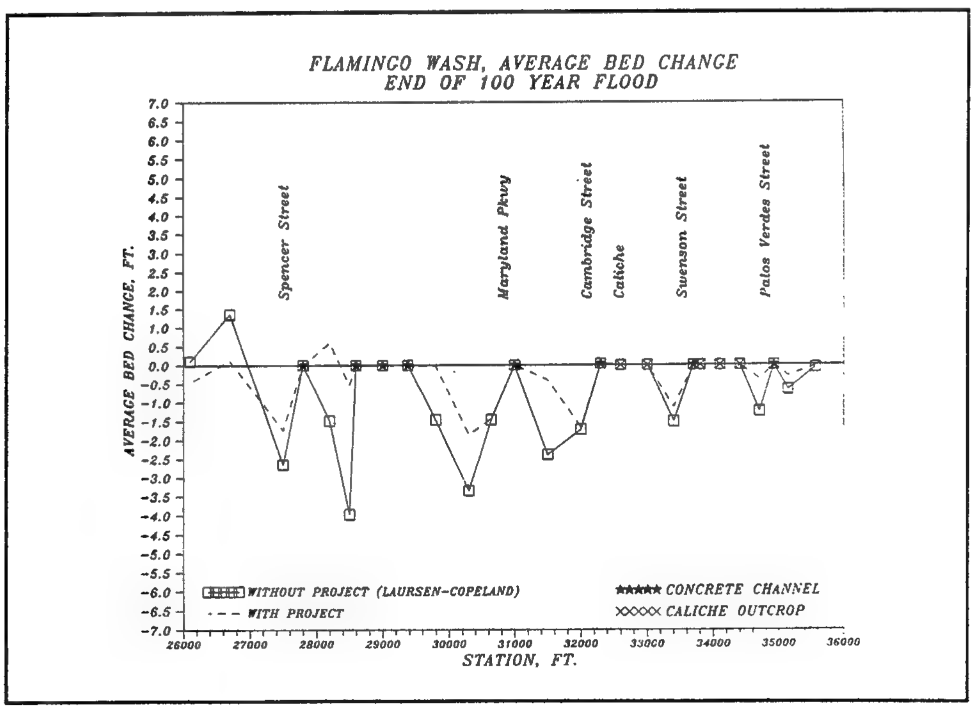


b. End of the 1-percent-chance exceedance hydrograph

Figure 33. Calculated average bed changes, Tropicana Wash stations 0+00 to 73+00

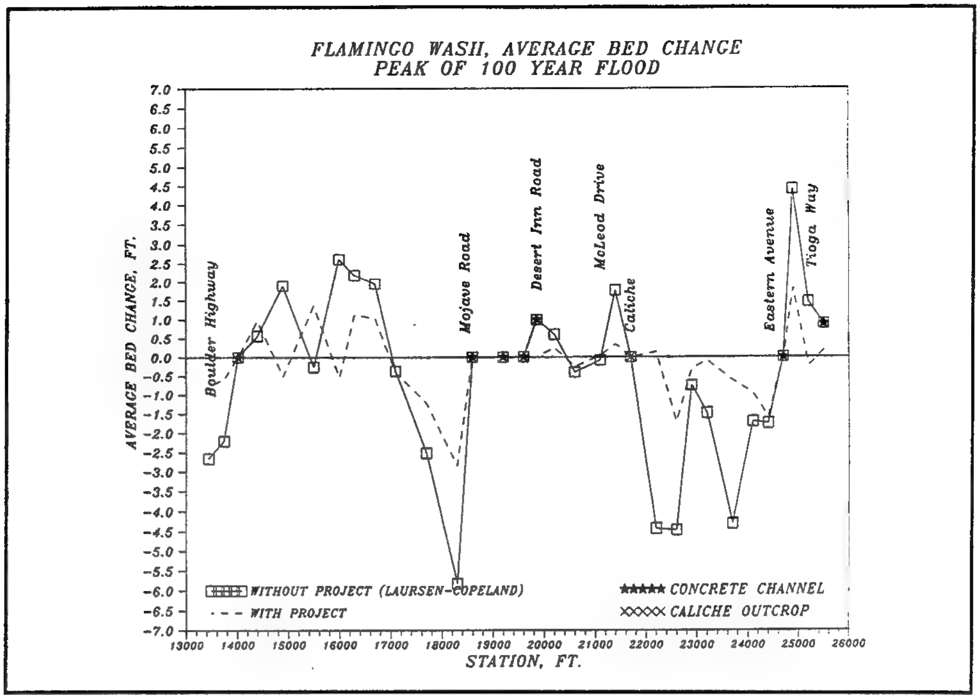


a. Peak of the 1-percent-chance exceedance hydrograph

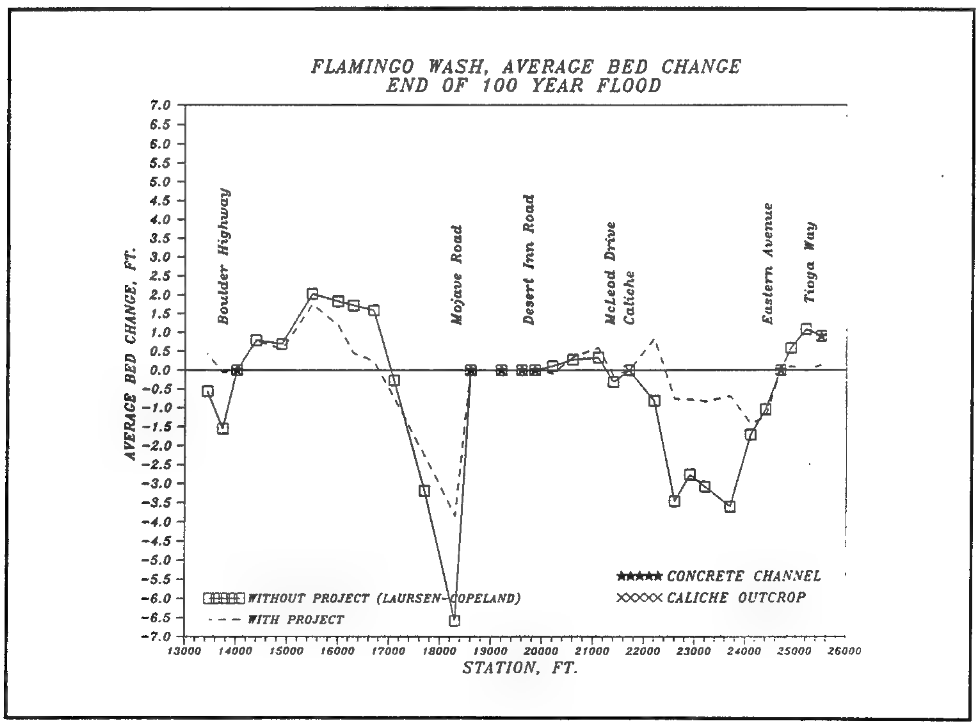


b. End of the 1-percent-chance exceedance hydrograph

Figure 34. Calculated average bed changes, Flamingo Wash stations 260+00 to 358+00

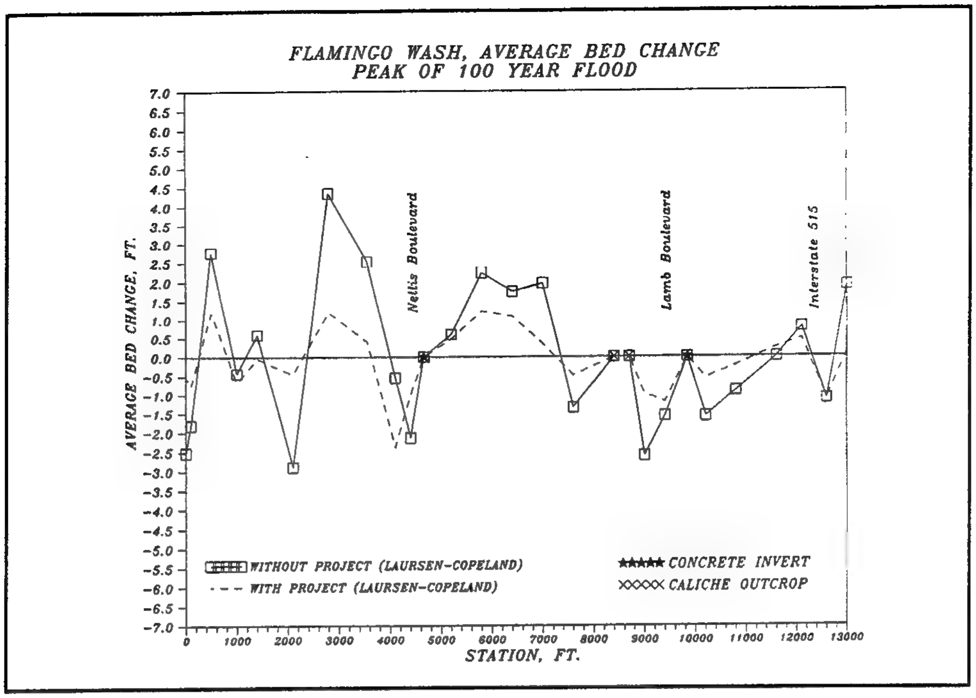


a. Peak of the 1-percent-chance exceedance hydrograph

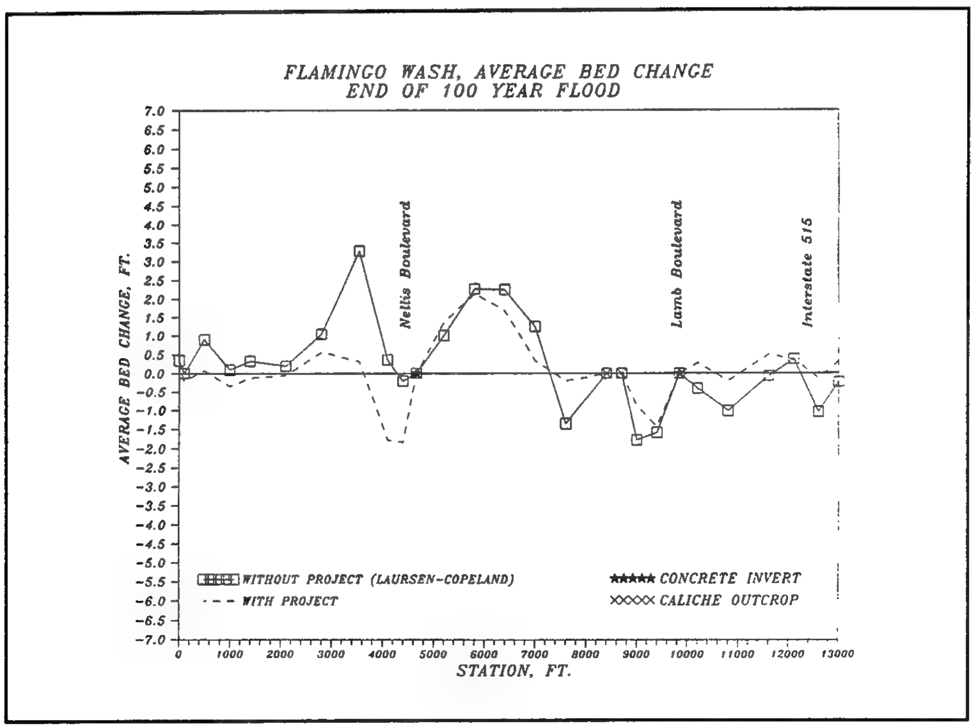


b. End of the 1-percent-chance exceedance hydrograph

Figure 35. Calculated average bed changes, Flamingo Wash stations 130+00 to 260+00

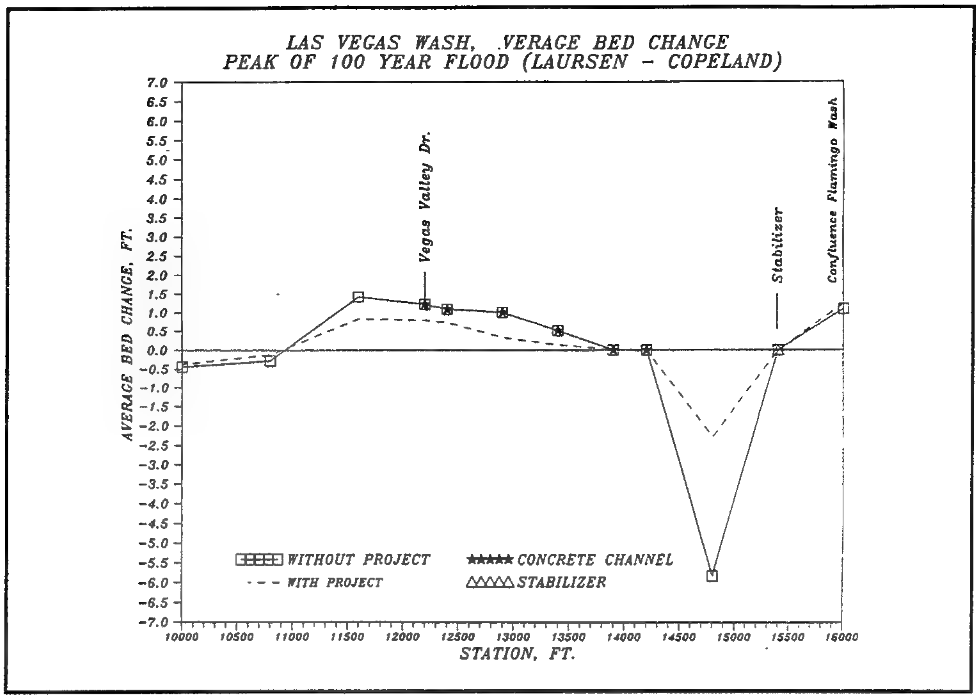


a. Peak of the 1-percent-chance exceedance hydrograph

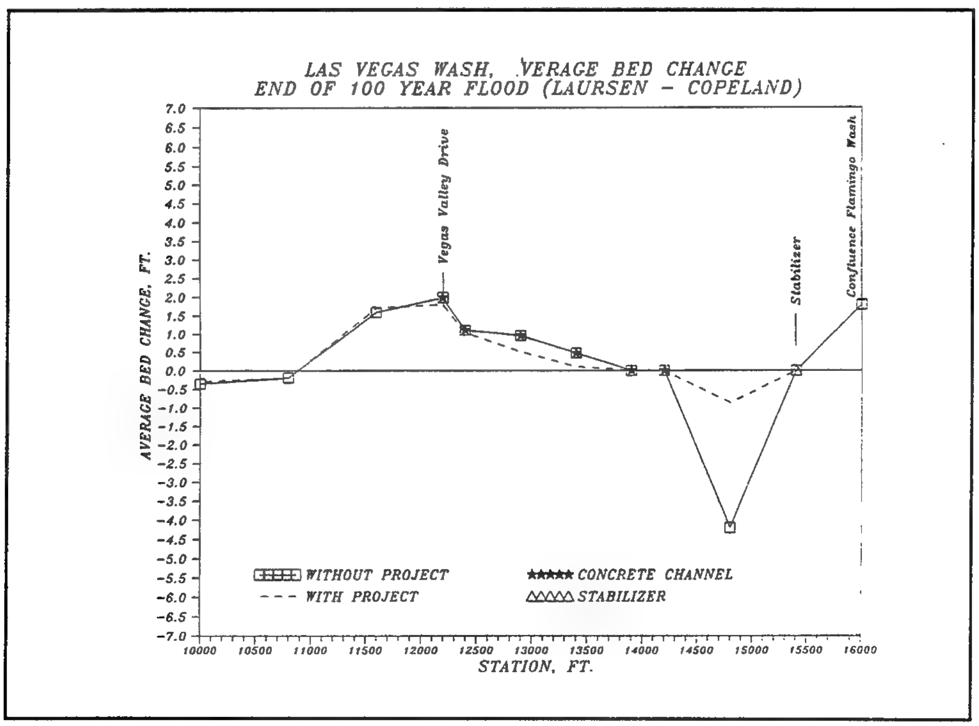


b. End of the 1-percent-chance exceedance hydrograph

Figure 36. Calculated average bed changes, Flamingo Wash stations 0+00 to 130+00



a. Peak of the 1-percent-chance exceedance hydrograph



b. End of the 1-percent-chance exceedance hydrograph

Figure 37. Calculated average bed changes, Las Vegas Wash stations 100+00 to 160+00

invert in place. The extent of the caliche deposit just downstream from Flamingo Road is uncertain; in the numerical model degradation was limited to two feet. The numerical model results indicate that the project will improve vertical channel stability on Tropicana Wash.

Caliche deposits are prevalent on Flamingo Wash between Palos Verdes Street and Cambridge Street. Potential for degradation is high in this reach, but in the numerical model bed scour was limited to two feet or not allowed at all. Due to the stabilizing effect of the caliche, calculated bed changes in this reach were small, with slightly less degradation calculated for the with-project condition.

Significant degradation was calculated on Flamingo Wash between Cambridge Street and Spencer Street. The bed elevation was maintained by concrete inverts at Maryland Parkway, Spencer Street, and through a short concrete-lined channel reach. Calculated degradation was significantly less for the with-project condition. A comparison of Figures 34a and b demonstrates that most of the degradation occurred by the time the peak flow occurred, and that little bed change occurred on the flood recession.

Some of the sediment scoured from the channel bed upstream deposited in the golf course reach between Spencer Street and Eastern Avenue. Calculated aggradation was greater at the flood peak than at the end of the flood. Aggradation was also more severe for the without-project condition than for the with-project condition.

Downstream from Eastern Avenue calculated degradation was significantly greater for the without-project condition than for the with-project condition all the way to McLeod Drive. This reach had recently been stripped of vegetation and reshaped. A caliche outcrop controls the bed elevation just upstream from McLeod Drive. Most of the degradation occurred by the time the peak discharge occurred for both without- and with-project conditions.

Flamingo Wash between McLeod Drive and Mojave Road was relatively stable for both without- and with-project conditions. Some aggradation was calculated upstream from Desert Inn Road at the peak discharge, but most of this deposit was removed by the end of the flood. The concrete-lined reach between Desert Inn and Mojave Roads remained free of deposits for all conditions tested.

The reach of Flamingo Wash downstream from Mojave Road was subject to significant degradation for all conditions tested. Calculated degradation was much greater for the without-project condition. Local clear water inflows at Desert Inn Road contribute to the severe erosion potential in this reach. Material scoured from the channel downstream from Mojave Road deposits in Flamingo Wash as it passes through a trailer park upstream from Boulder Highway. In this aggradational reach, more aggradation is calculated for the without-project condition.

Degradation was calculated immediately downstream from Boulder highway for both the without- and with-project hydrographs, although the degradation was much worse for the without-project condition. The general trend between Interstate 515 and Lamb Boulevard was calculated to be degradation for the without-project condition, but the bed was relatively stable for the with-project condition.

Downstream from Lamb Boulevard a potentially high degradation trend is damped by caliche outcrops. About halfway to Nellis Boulevard, the trend changes to an aggradation trend. Again, calculated bed changes are more significant for the without-project condition at the peak. However, by the end of the flood, the average bed elevation changes are about the same.

Downstream from Nellis Boulevard the calculated general trend is for aggradation through the golf course reach. At the peak of the flood, however, alternating aggradation and degradation are predicted by the numerical model in this reach for both the without- and with- project conditions. At the end of the flood, significant aggradation remains for the without-project condition, but degradation is calculated downstream from Nellis Boulevard for the with-project condition. This is one of the few reaches where the with-project condition could be viewed as less favorable than the without-project condition.

Calculated average bed changes in Las Vegas Wash downstream from its confluence with Flamingo Wash indicate significant degradation downstream from a stabilizer and then aggradation in the concrete channel to Vegas Valley Drive and beyond. The numerical simulation did not include any significant flow contribution from Las Vegas Wash upstream from its confluence with Flamingo Wash, so these calculations do not represent a one-percent chance exceedance flood on Las Vegas Wash. As with other reaches on Flamingo and Tropicana Washes, calculated average bed changes, both aggradational and degradational, are more severe for the without-project condition.

Sensitivity of Results to Transport Function

The results of the numerical simulations, reported above, were based on use of the Laursen-Copeland sediment transport function. This function was developed for use in sand and gravel-bed streams that have significant bed-material transport of both suspended and bed load. The function typically calculates a significantly higher load than equations developed for bed load transport only. There are no measured data to assess the reliability of any given transport equation for this project, so a sensitivity study was conducted to determine if use of a different transport function would produce different conclusions. The Meyer-Peter and Muller equation, which was developed for bed load transport in gravel bed streams, was used in the sensitivity study.

Average-bed elevation changes were calculated for the without-project condition using the same geometric and hydrologic model components in the

HEC-6W numerical model. Calculated results at the hydrograph peaks are compared to calculated results using the Laursen-Copeland sediment transport function in Figures 38-40. The calculated degradation and aggradation trends were found to be essentially identical using the two equations. However, as expected, the magnitude of the calculated degradation and aggradation is less when the Meyer-Peter and Muller equation is employed. The sensitivity study demonstrates the level of reliability that can be expected from the un-circumstantiated numerical model. The calculated depths of scour and deposition should not be considered as design values. However, a reasonable range of expected values can be inferred.

The purpose of the numerical simulation was to evaluate the relative difference in degradation and aggradation potential for the without-project and with-project hydrographs. This qualitative assessment may be made even though sediment transport functions produce different magnitudes of calculated scour and deposition.

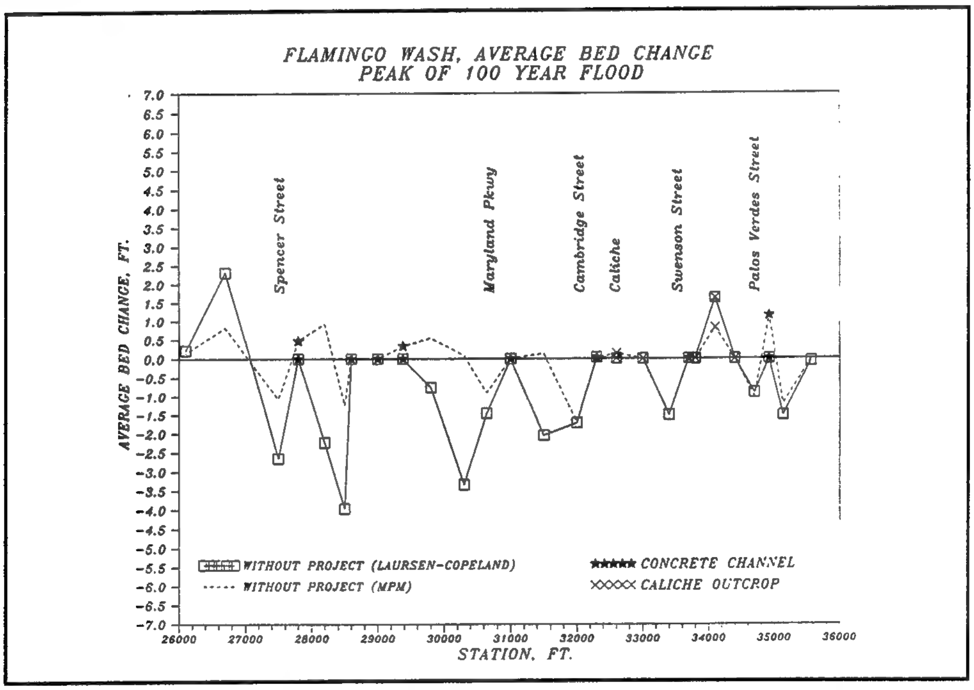


Figure 38. Comparison on Sediment Transport Equations - calculated average bed changes - Flamingo Wash Station 260+00 to 358+00 at the peak of the one-percent-chance exceedance hydrograph

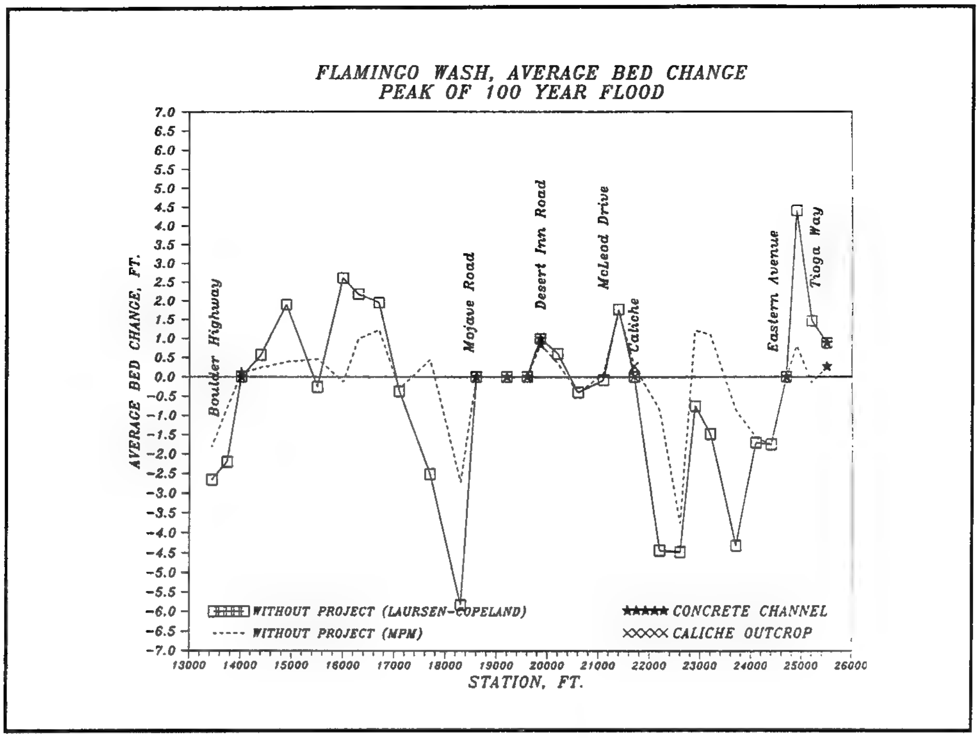


Figure 39. Comparison on Sediment Transport Equations - calculated average bed changes - Flamingo Wash Station 130+00 to 260+00 at the peak of the one-percent-change exceedance hydrograph

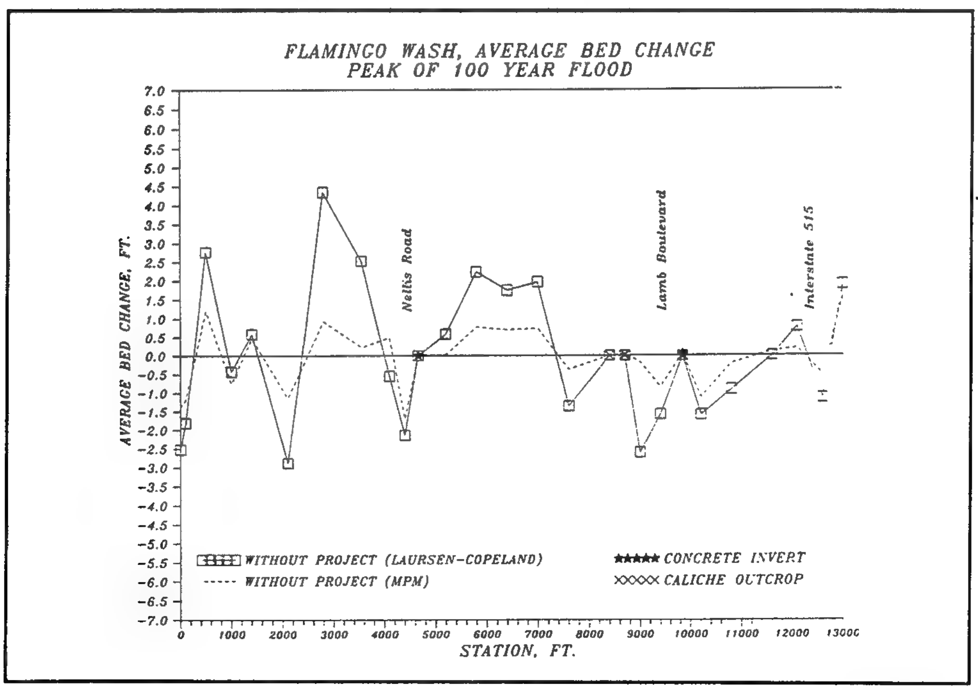


Figure 40. Comparison on Sediment Transport Equations - calculated average bed changes - Flamingo Wash Station 0+00 to 130+00 at the peak of the one-percent-chance exceedance hydrograph

5 Damage Potential Assessment

The damage potential assessment is based on field observations of the channel described in the channel inventory, calculated aggradation and degradation potentials, and the potential for damage due to the proximity of structures to the channel bank. Three categories were established: proximity of structures to the bank, bank instability, and potential for scour or deposition. The assessment is qualitative. In each category, general levels of intensity - high, medium, or low - were assigned to each reach or portion of a reach. The reaches, or portions of a reach, that were assigned a high potential in all three categories would be the reaches most likely to have damage caused by channel instability. The results are shown in Tables 3 and 4.

Distance of structures to the top bank was determined using the 1993 aerial photographs and topographic maps. If structures were more than 100 ft from the channel top of bank then potential damage was classified as low. If structures were between 51 and 100 ft from the channel top bank then potential for damage was classified as medium. Structures within 50 ft of the channel top bank were classified as having a high potential for damage.

Damage potential due to bank instability potential was estimated based on field observations and classified as low, medium or high. Channel reaches that were underground or fully concrete-lined were assigned a low erosion potential. In the medium category were shaped trapezoidal reaches with concrete-lined banks, continuous rubble banks, and vegetated reaches. High damage potential areas were assigned to reaches with no bank protection. A more detailed study would be required to obtain a more reliable estimate of bank erosion potential. This would include field testing to determine the geotechnical characteristics of the bank material. In addition to detailed geometric definition of the natural bankline, soil properties of cohesion, friction angle, and specific weight would be required. If the bank is composed of layers of variable material, the analysis becomes considerably more complicated. The level of bank protection provided by the dumped rubble that was observed throughout the entire length of the study reach is uncertain. Detailed analysis of the riprap stability may be appropriate at critical locations.

Potential for scour or deposition was classified as low, medium or high based on the numerical model results. Reaches with calculated degradation or aggradation less than one ft were considered to have a low potential for instability. Reaches with calculated degradation or aggradation greater that two feet were considered to have a high potential for instability. The classification was based on calculated results for the without-project hydrograph. As can be seen in Figures 33-37, some reaches would have a lower potential for instability with the project in place.

The damage potential assessment is contained in Tables 3 and 4. Reaches identified as having the highest damage potential were: the right bank of reach 8, between Maryland Parkway and the concrete-lined channel; the right bank of reach 10, just downstream from Eastern Avenue; both banks of reaches 13 and 14 between Mojave Road and Boulder Highway; and the right bank of the downstream half of reach 17, upstream from Nellis Boulevard. These reaches should be carefully monitored during flood events. This analysis does not preclude other reaches from suffering damage during flood events, or even from suffering more damage than the identified reaches.

6 Channel Slope Stability

Observations and judgement were used to assess channel slope stability for the channel inventory. A more detailed methodology is presented in *Channel Stability Assessment for Flood Control Projects* - EM 1110-2-1418 (USACE 1994). This engineering manual presents techniques suitable for use during the early stages of planning studies. The approach outlined in the engineering manual is appropriate for evaluating the general channel stability of the existing Flamingo and Tropicana Wash channels and to make preliminary estimates of required dimensions for a stable channel design. Methodologies applied to the Flamingo and Tropicana Wash study reaches are the allowable velocity method and the stable channel analytical method.

Allowable Velocity

The allowable velocity method is a threshold method based on the premise that average channel velocity is a suitable parameter to define channel stability and that threshold velocities can be identified for a given type of boundary material. Tables of maximum allowable velocities are given in a number of sources. EM 1110-2-1418 provides a table of suggested maximum permissible channel velocities for several types of channel materials. Those applicable to Flamingo and Tropicana Washes are tabulated.

Channel Boundary Material	Permissible Mean Channel Velocity, ft/sec
Fine gravel (5 - 20 mm)	6.0
Poor Rock (usually sedimentary)	10.0
Bermuda grass - on sandy silt - on silt clay	6.0 8.0
Kentucky Blue Grass - on sandy silt - on silt clay	5.0 7.0

Caliche may be classified as poor rock, and a maximum permissible velocity of 10 ft/sec can be assigned, although the resistivity of different caliche deposits is

expected to vary significantly. Resistivity of grass-lined channels is a function of the type of grass and the composition of the sub-surface soil. However, neither the existing grass types nor the subsurface soils were identified during the field reconnaissance. A representative maximum permissible velocity for grass-lined channels of 6 ft/sec was selected for this evaluation. The movable bed portions of Flamingo and Tropicana Washes consist of sands and gravels, with a median grain diameter of 6.5mm. A maximum permissible velocity of 6 ft/sec is suggested for fine gravel. Other techniques presented in EM 1110-2-1418 suggest different maximum allowable velocities for granular material. Figure 5-5 (USACE 1994) suggests a maximum allowable velocity of about 4 ft/sec for a median grain size of 6.5 mm at a depth of 5 ft. Tropicana and Flamingo Washes have a d_{75} grain size of 18 mm, for which Figure 5-1 (USACE 1994) suggests a basic maximum allowable velocity between 4.5 and 6.5 ft/sec for sediment free (< 1,000 ppm) and sediment laden (> 20,000 ppm) flow respectively. Figure 5-1 is taken from U.S. Soil Conservation Service design criteria for open channels (USDA 1977). The USDA method includes adjustments for alignment, bank slope, and depth. When these adjustments are accounted for in a typical reach of Flamingo Wash, the range of maximum permissible velocities is between 3.8 and 6.5 ft/sec. These adjustments apply to a relatively straight reach (i.e. where the ratio of radius of curvature to water surface width is greater than 14); with bank slope angles which vary between 1V:2H and 1V:3H; and with an average depth of 6.6 ft, which is the average calculated depth for the one-percent-chance exceedance peak flow in Flamingo Wash. Considering the various recommended values for the granular bed material, an average maximum permissible velocity for unlined channels of 5 ft/sec was selected for this evaluation.

Average channel velocities were calculated for the with-project one-percent-chance exceedance peak discharge of 6,600 cfs, the ten-percent-chance exceedance peak discharge of 2,900 cfs, and 400 cfs, which is the discharge that carries the most sediment during the with-project one-percent-chance exceedance hydrograph. Channel velocities were calculated using the initial channel geometry from the HEC-6W numerical model. These are compared in Figures 41-44 with allowable velocities of 6 ft/sec in grass-lined channels and 5 ft/sec in unlined channels. No consideration is given in this preliminary evaluation for existing rubble riprap or concrete-lined side slopes, because the bed is left unprotected. Caliche outcrops were not given consideration because the banks are left unprotected. Maximum permissible velocities would be less in reaches with significant channel curvature. Maximum permissible velocities would be greater if sufficient caliche deposit control can be confirmed.

The comparison confirms the conclusions reached in the channel inventory - that during major flood peak discharges, the existing channel is unstable in all reaches unless the channel is fully concrete-lined. At 400 cfs, the maximum allowable velocity is exceeded at about 40 percent of the cross sections. The grass-lined and fully concrete-lined reaches would be considered stable at this lower discharge. At a discharge of 400 cfs, the maximum allowable velocity is generally not exceeded in the unlined channel downstream from Boulder

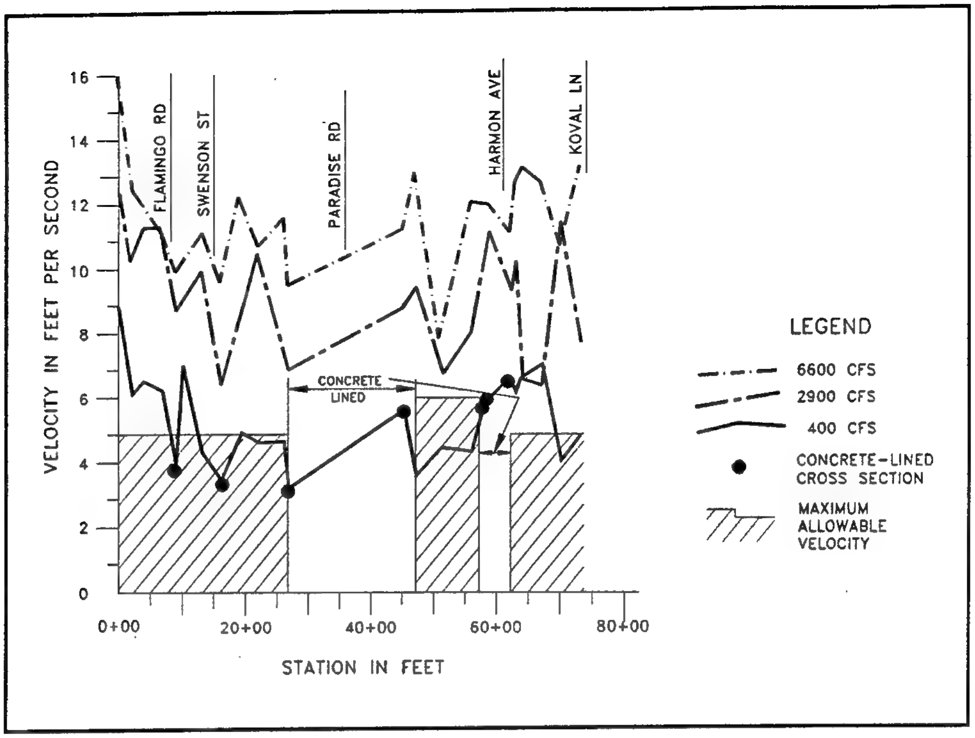


Figure 41. Maximum allowable velocities - Tropicana Wash, Station 0+00 to 73+00

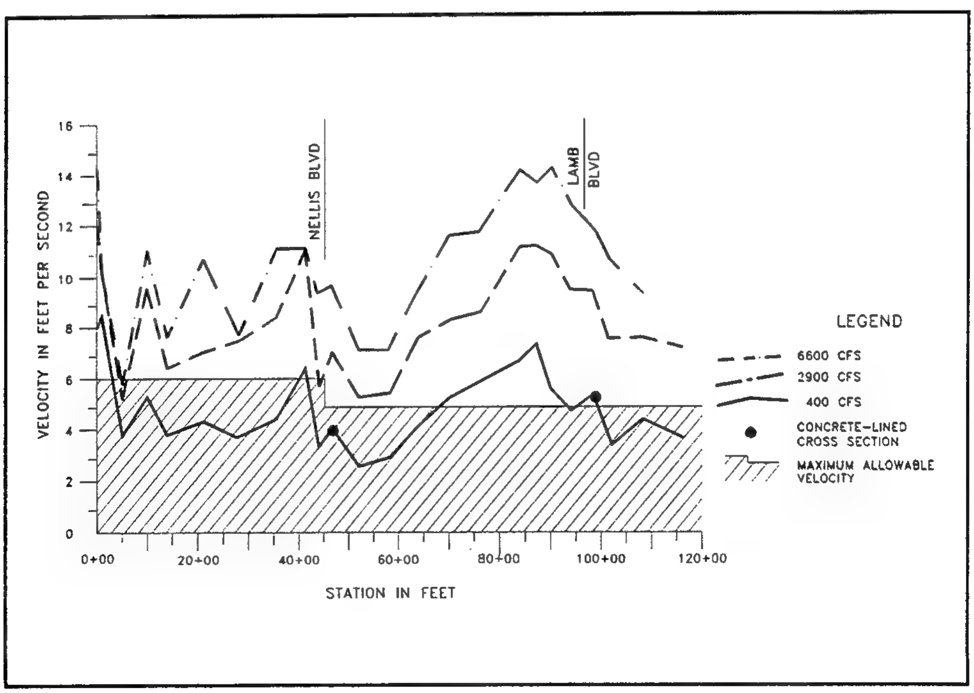


Figure 42. Maximum allowable velocities - Flamingo Wash, Station 0+00 to 116+00

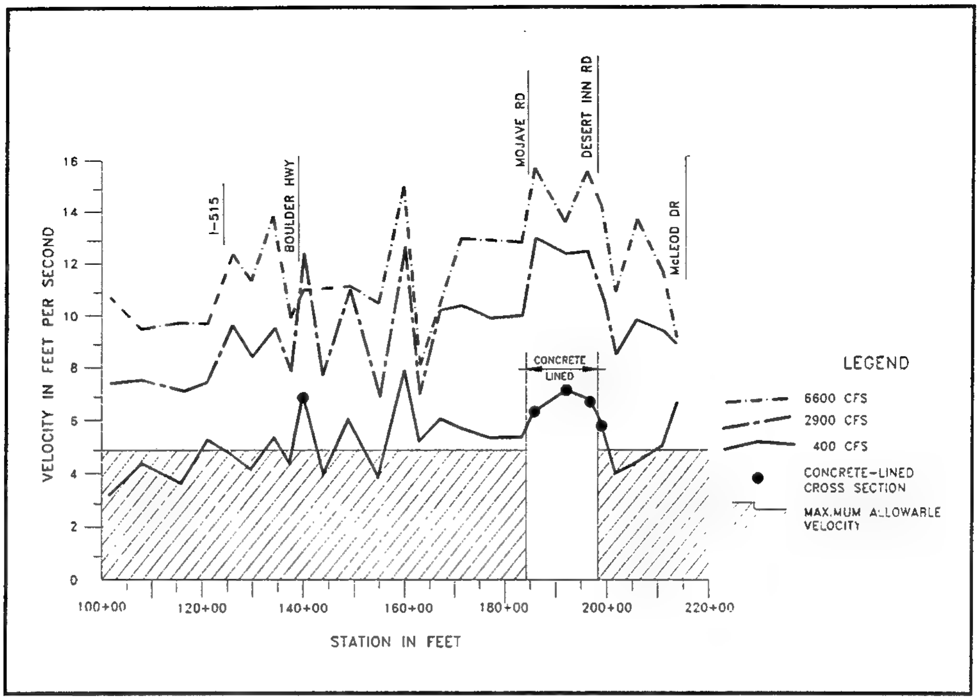


Figure 43. Maximum allowable velocities - Flamingo Wash, Station 102+00 to 217+00

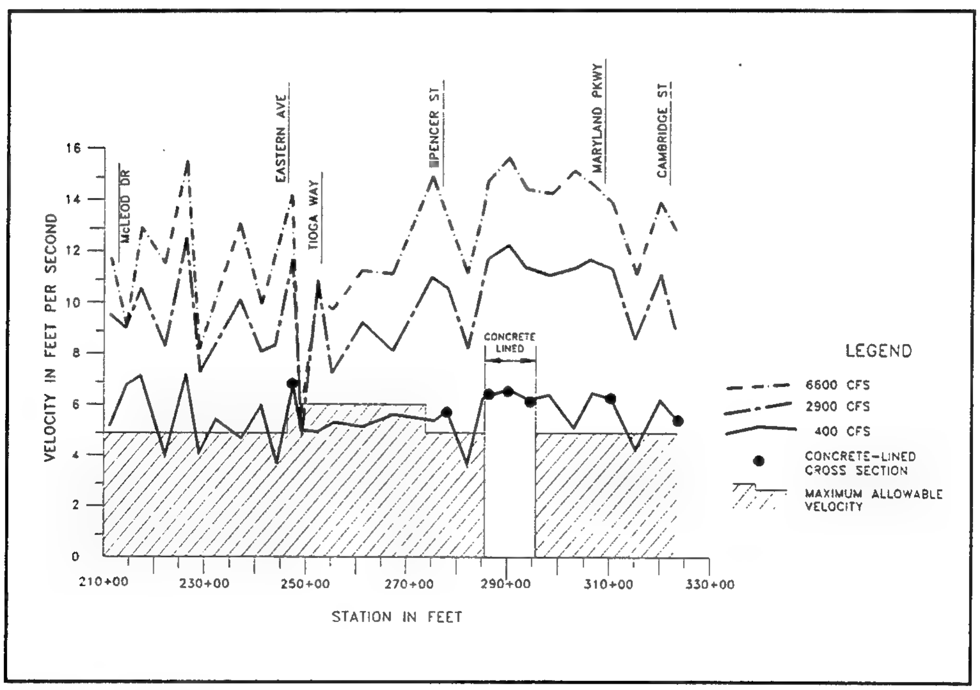


Figure 44. Maximum allowable velocities - Flamingo Wash, Station 211+00 to 323+00

Highway to Las Vegas Wash, except for the reach just downstream from Lamb Boulevard.

Stable channel design using the allowable velocity approach would require that velocities for the design channel be less than the maximum allowable value. In order to account for uncertainties and localized anomalies, it is not recommended that the channel be designed for velocities less than those expected at the peak of the ten-percent-chance exceedance flood. Required design slopes were calculated assuming normal depth for several possible design trapezoidal channel base widths. The proposed channel was assigned a side slope angle of 1V:2.5H, a bed roughness coefficient of 0.035, and a bank roughness coefficient of 0.040. Calculated channel design dimensions are given in the following tabulation.

Channel Dimensions Calculated using Allowable Velocity Approach
Unlined channels

Discharge cfs	Velocity ft/sec	Cotangent of Bank Slope	Base Width ft	Water Depth ft	Bed Slope
2900	5.0	2.5	25	11.0	0.0013
2900	5.0	2.5	50	8.2	0.0014
2900	5.0	2.5	75	6.4	0.0016
2900	5.0	2.5	100	5.2	0.0019

The calculated design channel slopes are significantly less that the existing average channel slope of 0.0055.

Stable Channel Analytical Method

The stable channel analytical method (Copeland 1994) is an analytical technique to calculate channel dimensions that will transport an incoming bed-material sediment load, at a specified discharge, with no aggradation or degradation. It numerically solves a sediment transport equation and resistance equation, providing a family of width and slope solutions that satisfies the equations. The method uses the Brownlie (1981) equations to calculate sediment transport and channel bed roughness. These equations were developed for sand-bed streams, and therefore results must be used with caution. Further, as discussed in Chapter 3, the non-alluvial characteristics of Flamingo and Tropicana Washes makes application of any sediment transport equation questionable, and results should be interpreted qualitatively. Results of the stable channel method are adequate for obtaining a general idea of required stable channel dimensions for a planning level study.

The stable channel analytical method is intended for use in streams with bed-material load. When the Tropicana detention basin is constructed, bed-material supply will be cut off in Tropicana Wash. A decrease in

bed-material supply is also expected in Flamingo Wash due to the redistribution of flow on the upper alluvial fans by the project features. Therefore, the method is not applicable to Tropicana Wash directly downstream from the proposed dam. However, immediately after construction, downstream degradation and bank erosion may provide a source to re-establish a bed-material load further downstream. The HEC-6W numerical model study indicated that reach 11, between Desert Inn Road and McLeod Drive, would be relatively stable during the one-percent-chance exceedance flood. This indicates that an equilibrium sediment transport condition had been established in this reach during the simulation. The stable channel analytical method was applied to reaches downstream from reach 11, using the calculated bed-material sediment load in reach 11 as the upstream boundary condition.

Reach 11 was selected as the supply reach because it was found to be subject to neither significant aggradation or degradation during the numerical model study. A typical trapezoidal cross-section was developed for this reach based on the HEC-6W model geometry. Channel banks, which were partially covered by rubble, were assigned a roughness coefficient of 0.050. The same bed-material gradation used in the numerical model was assigned to the bed. Energy slopes were determined for discharges of 400; 1,000; 2,900; and 6,600 cfs from a HEC-2 backwater calculation using the initial HEC-6W model geometry.

The first reach downstream from reach 11 was a concrete-lined channel and required no analysis. The next reach, reach 13, between Mojave Road and Vegas Valley Drive, was a graded trapezoidal channel with a cut low-flow channel. The stable channel analytical method was applied to this reach to obtain a preliminary design estimate of stable channel dimensions. The proposed channel design was a trapezoidal channel that retained the unlined 1V:3H side slopes, which were assigned a roughness coefficient of 0.040. The stable channel calculations were made using the hydraulic design package SAM. The calculated stable channel design curves are shown in Figure 45. Also shown on this figure are the base width and slope ranges of the existing channel. The existing channel dimensions plot well above the stability curves for all discharges, indicating an existing channel regime subject to degradation. Since the existing channel passes through a residential neighborhood, it would be difficult to widen the channel. Therefore, retaining the existing channel base width of about 60 ft, a design channel slope of 0.0050 is obtained from the stability curves for 2,900 cfs. The estimated design slope is about 50 percent of the existing channel slope. An 11-ft drop would be required through the 2000-ft-long reach to attain this slope. Calculated water depth at 2,900 cfs is 5.8 ft, average channel velocity is 6.4 ft/sec, and the composite roughness coefficient is 0.046. This high roughness coefficient is the result of bed-form roughness calculated by the Brownlie equations. The discontinuity in the stability curve for 6600 cfs is due to transition from upper regime plane bedforms to lower regime dune bedforms.

The stable channel method was applied to the next reach downstream, reach 14, located between Vegas Valley Drive and Boulder Highway. The existing channel in this reach is generally trapezoidal in shape with irregular 1V:2H bank slopes. The design channel was assumed to retain the same shape as the existing channel. The banks were assigned a roughness coefficient of 0.050. For design purposes, sediment inflow was assumed to come from reach 11. The calculated stability curves are shown in Figure 46. Also shown are the ranges of slope and base width in the existing channel. The existing channel dimensions fall above the stability curves for all discharges except the one-percent-chance exceedance peak. This indicates that the channel would be relatively stable during the peak of the one-percent-chance exceedance hydrograph but would be subject to degradation at the other discharges. Using an average channel base width of 30 ft, which matches the existing channel, a design slope of 0.0053 is suggested for 2,900 cfs. This is only a 15 percent decrease from the existing channel slope. A 1.5-ft drop would be required in this 1600-ft-long reach to attain this slope. Calculated water depth at 2,900 cfs is 8.7 ft, average channel velocity is 7.0 ft/sec, and the composite roughness coefficient is 0.051. This high roughness coefficient is the result of bed-form roughness calculated by the Brownlie equations.

Reach 15, between Boulder Highway and Interstate-515 was not evaluated because significant channel reshaping activity has occurred since the channel geometry was defined in 1993.

Between Interstate-515 and Lamb Boulevard is reach 16, an unlined trapezoidal channel with 1V:3H side slopes. The same channel shape was assumed for the stable channel design. A bank roughness coefficient of 0.040 was assigned. Reach 16 has the same stability curves as reach 13 because the channel side slopes are the same and the sediment supply reach is the same. The stability curves are shown in Figure 47 with the existing channel ranges of slope and base width. The stability curves indicate that the existing channel should be stable for all discharges, except the one-percent-chance exceedance discharge. At a discharge 6,600 cfs, aggradation is predicted. The analysis suggests that the existing slope of about 0.0052 and base width of 75 ft be retained. For a discharge of 2,900 cfs, calculated water depth is 5.2 ft, average channel velocity is 6.1 fps, and the composite roughness coefficient is 0.047.

Channel stability analyses were not conducted for reaches 17-20. Reach 17 has been significantly reshaped since the topographic data was obtained. Reach 18 is a grass swale through a golf course. Reaches 19 and 20 include Las Vegas Wash where upstream flow and sediment transport were not considered in this study.

The stable channel method only provides for sediment transport capacity through the reaches. In all the reaches studied herein, the calculated channel velocity was higher than the maximum permissible velocities. Therefore, bank protection would be required for all these stable channel designs.

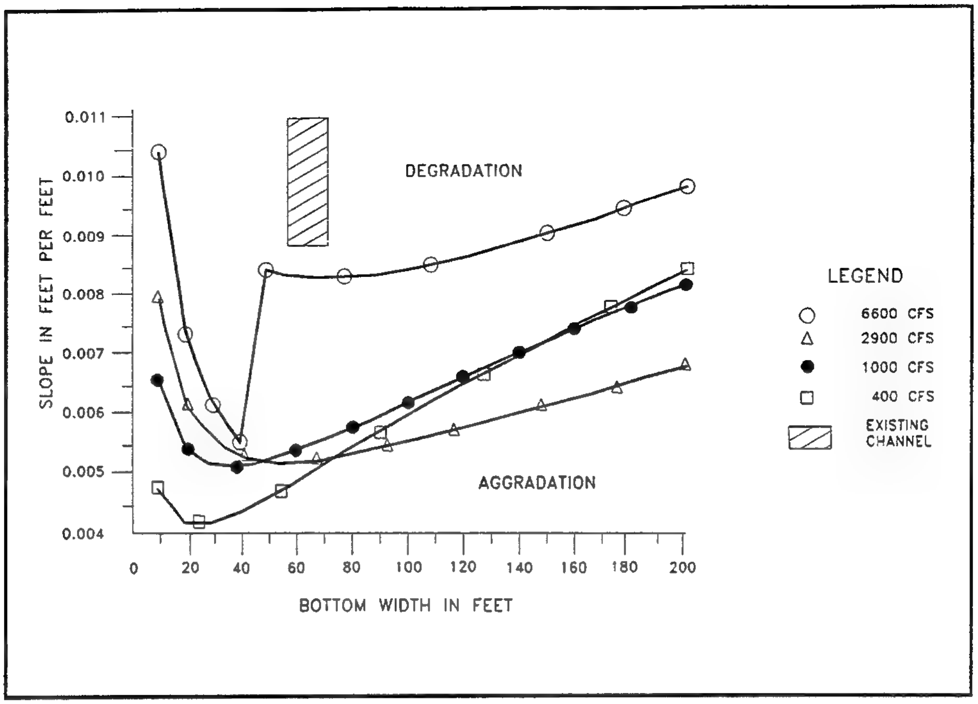


Figure 45. Channel stability curve - Flamingo Wash, Reach 13

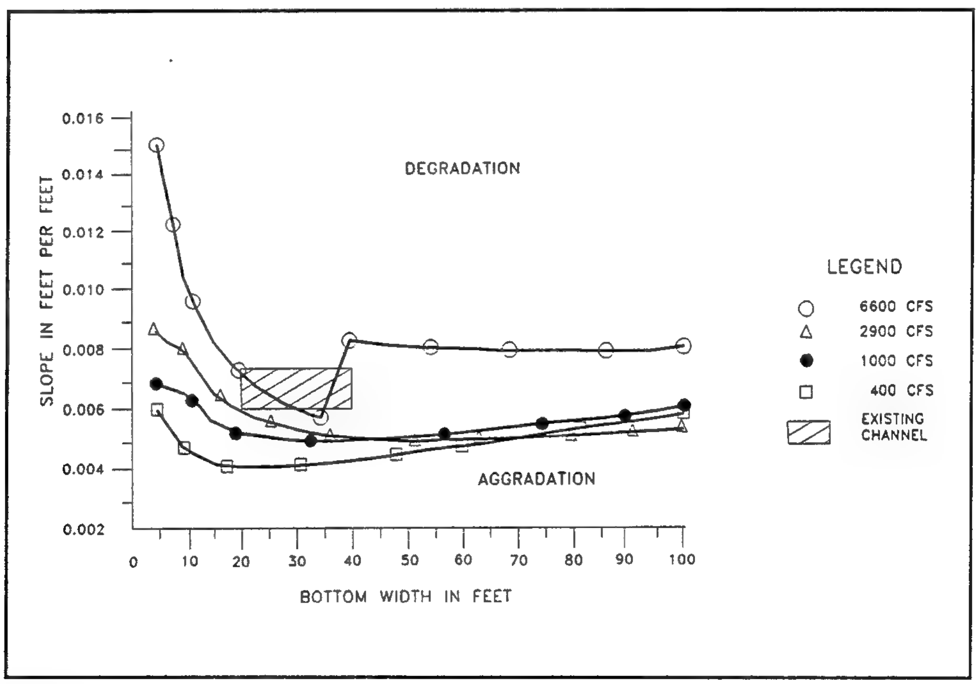


Figure 46. Channel stability curve - Flamingo Wash, Reach 14

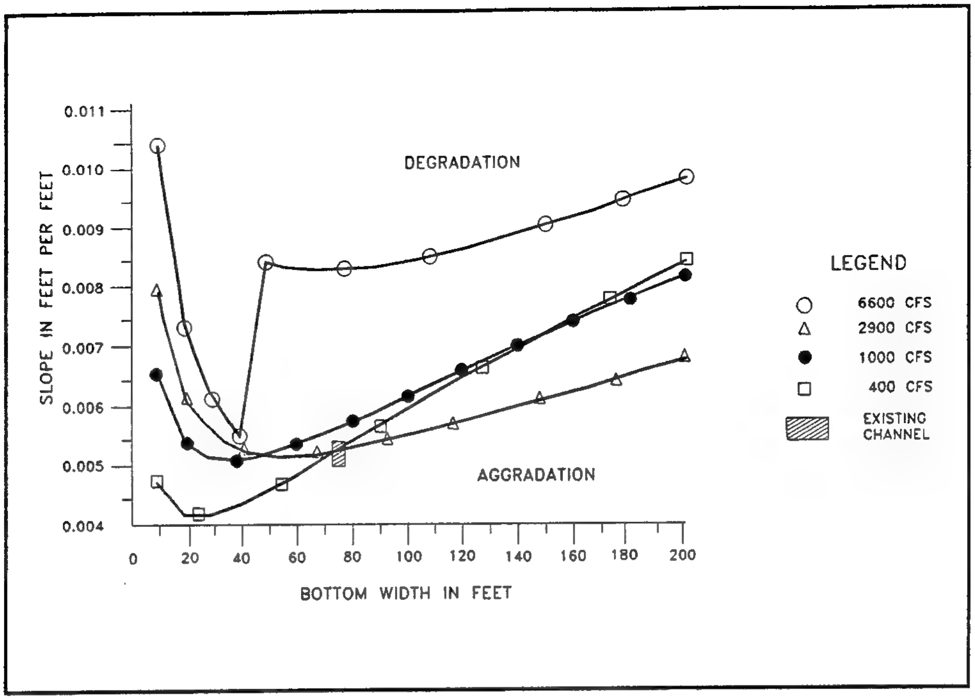


Figure 47. Channel stability curve - Flamingo Wash, Reach 16

Summary

Stable channel slope calculations provide general guidance for estimating design slopes for the Flamingo and Tropicana channels downstream from the proposed Tropicana detention basin for planning purposes. To prevent both bank and bed erosion, the allowable velocity method suggests that a design slope between 0.0013 and 0.0019 would be required. Bank protection would be required where channel curvature is present. Assuming that sufficient sediment is supplied from upstream, the stable-channel method suggests that a design slope, downstream from Mojave Road, of between 0.0050 to 0.0055 would be required to maintain vertical stability in the channel bed. Bank protection would be required in all reaches.

7 Conclusions and Recommendations

The channel stability assessment for Flamingo and Tropicana Washes was conducted to assess the potential for change in channel stability associated with the implementation of a proposed Corps of Engineer flood control plan for the City of Las Vegas. The flood control plan included flood detention basins that would store the peak flood flows and release them at a lower flow rate over a longer period of time.

A channel inventory was conducted to identify channel characteristics that would affect channel stability. Degradation in the existing Tropicana and Flamingo Wash channels is checked by numerous bed controls, both man-made and natural. The most severe zones of degradation were found to be downstream of man-made controls, especially road culverts and concrete-lined channels. Most channels are graded trapezoidal channels, with a cut low-flow channel. Although bank protection has been placed along the channel banks and the low-flow channels in many reaches, there remain many channel reaches with little or no bank protection. Vegetation has been systematically removed from the channel banks and beds, eliminating this stabilizing natural feature. Bank erosion is expected throughout the study reach for both the with-project and without-project one-percent-chance exceedance floods.

A numerical model simulation was conducted to assess the difference in the vertical stability of the existing channel for the with-project and the without-project one-percent-chance exceedance hydrographs. It was determined that degradation and aggradation would be roughly twice as severe for the without-project hydrograph. This result was consistent throughout the study reach. Since bank erosion potential can be related to the magnitude of vertical instability it is concluded that both bank erosion and vertical channel instability will be less severe with the flood control project in place.

Aerial photographs, topographic maps, and field investigations were used to establish the proximity of structures to the existing channel bank. These results were combined with results from the channel inventory and the numerical simulation to make a qualitative determination of the which reaches were the

most likely to suffer damage during a flood event. It is recommended that these reaches be carefully monitored, especially after floods.

References

Brownlie, William R. 1981. *Prediction of Flow Depth and Sediment Discharge in Open Channels,* Report No. KH-R-43A, California Institute of Technology, W.M. Keck Laboratory of Hydraulics and Water Resources, Pasadena, CA.

Copeland, Ronald R. 1994. *Application of Channel Stability Methods - Case Studies,* Technical Report HL-94-11, U.S. Army Engineer Waterways Experiment Station, Hydraulics Laboratory, Vicksburg, MS.

__________ and Thomas, W. A. 1989. *Corte Madera Creek Sedimentation Study; Numerical Mode Investigation,* Technical Report HL-89-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________, McComas, D.N., Raphelt, N.K., and Thomas, W.A. 1996. *User's Manual for the Hydraulic Design Package for Channels (SAM),* U.S. Army Engineer Waterways Experiment Station, Vicksburg MS.

Einstein, Hans A. 1950. "The Bed-Load Function for Sediment Transportation in Open Channel Flow," Technical Bulletin No. 1026, U.S. Department of Agriculture, Soil Conservation Service, Washington D.C.

James M. Montgomery, Consulting Engineers, Inc, 1988. *Las Vegas Wash and Tributaries Overflow Study, Clark County, Nevada,* Technical report prepared for the U.S. Army Corps of Engineers, Los Angeles District and for the Clark County Regional Flood Control District.

Meyer-Peter, E. and Muller, R. 1948. "Formulas for Bed-Load Transport," *Second Meeting of the International Association for Hydraulic Structures Research,* Stockholm, Sweden, appendix 2, pp34-64.

Toffaleti, F.B. 1968. *A Procedure for Computation of the Total River Sand Discharge and Detailed Distribution, Bed to Surface,* Technical Report Number 5, Committee on Channel Stabilization, U.S. Army Corps of Engineers, printed at U.S. Army Waterways Experiment Station, Vicksburg, MS.

U.S. Army Corps of Engineers (USACE) 1994. *Channel Stability Assessment for Flood Control Projects* - EM 1110-2-1418, Washington D.C.

U.S. Army Engineer District (USAED), Los Angeles 1991a. *Feasibility Report and Environmental Impact Statement: Las Vegas Wash and Tributaries (Tropicana and Flamingo Washes), Nevada.* Hydraulics Technical Documentation Report.

U.S. Army Engineer District (USAED), Los Angeles 1991b. *Feasibility Report and Environmental Impact Statement: Las Vegas Wash and Tributaries (Tropicana and Flamingo Washes), Nevada.*

U.S. Army Engineer District (USAED), Los Angeles 1991c. *Feasibility Report and Environmental Impact Statement: Las Vegas Wash and Tributaries (Tropicana and Flamingo Washes), Nevada.* Hydrology Technical Documentation Report.

U.S. Army Engineer District (USAED), Los Angeles 1993. *Las Vegas Wash and Tributaries (Tropicana and Flamingo Washes),* Hydrologic Basis for Design.

U.S. Army Engineer Hydrologic Engineering Center (USAEHEC) 1993. *User's manual HEC-6 generalized computer program for scour and deposition in rivers and reservoirs.* Davis, CA.

U.S. Department of Agriculture (USDA) 1977. Design of Open Channels, Technical Release No. 25. Soil Conservation Service, Washington D.C.

Table 1
Tropicana Wash Average Bed Change

Channel Station No.	Peak of 100 Year Flood: Without Project	Peak of 100 Year Flood: With Project	End of 100 Year Flood: Without Project	End of 100 Year Flood: With Project
7300	-0.20	-0.13	-0.20	-0.10
7001	-1.37	-1.20	-1.37	-1.34
6700	-5.30	-2.51	-5.04	-3.87
6401	-1.99	-0.79	-1.59	-1.50
6300	0.47	0.28	-0.42	-1.44
6200	1.31	0.00	0.28	0.00
5860	0.00	0.00	0.09	0.00
5820	0.00	0.00	0.00	0.00
5600	0.16	0.35	0.48	0.06
5100	0.47	0.40	0.37	0.47
4700	-0.32	-0.03	0.04	0.21
4500	0.00	0.00	0.00	0.00
2650	0.00	0.00	0.00	0.00
2600	-1.29	-0.70	-1.58	-1.59
2200	-0.45	-0.41	-0.47	-0.39
1900	0.37	0.13	0.26	-0.02
1600	0.99	0.89	1.12	0.55
1300	-0.12	0.46	0.51	0.21
1000	0.00	0.00	0.00	0.00
880	0.00	0.00	0.00	0.00
660	-1.72	-1.53	-1.72	-1.69
400	0.00	0.00	0.00	0.00
200	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00

Table 2
Flamingo Wash Average Bed Change

Channel Station No.	Peak of 100 Year Flood: Without Project	Peak of 100 Year Flood: With Project	End of 100 Year Flood: Without Project	End of 100 Year Flood: With Project
32300	0.04	0.02	0.04	0.02
32000	-1.72	-1.63	-1.72	-1.66
31500	-2.05	-0.17	-2.39	-0.40
31000	0.00	0.00	0.00	0.00
30640	-1.46	-1.33	-1.46	-1.43
30300	-3.34	-1.25	-3.35	-1.85
29800	-0.76	0.30	-1.46	-0.02
29380	0.00	0.00	0.00	0.00
29000	0.00	0.00	0.00	0.00
28600	0.00	0.00	0.00	0.00
28500	-3.96	-0.07	-3.97	-0.53
28200	-2.23	0.59	-1.48	0.61
27800	0.00	0.00	0.00	0.00
27500	-2.65	-0.99	-2.65	-1.73
26700	2.32	0.72	1.36	0.13
26100	0.22	-0.30	0.11	-0.46
25500	0.88	0.18	0.90	0.14
25200	1.46	-0.23	1.08	-0.02
24900	4.40	1.83	0.59	0.11
24700	0.00	0.00	0.00	0.00
24400	-1.76	-1.60	-1.05	-1.25
24100	-1.71	-0.97	-1.71	-1.42
23700	-4.32	-0.63	-3.61	-0.69
23200	-1.49	-0.09	-3.09	-0.84
22900	-0.76	-0.32	-2.77	-0.78
22600	-4.48	-1.73	-3.46	-0.77
22200	-4.44	0.14	-0.82	0.85
21700	0.00	0.00	0.00	0.00
21400	1.77	0.34	-0.31	-0.14
21100	-0.08	0.05	0.34	0.60
20600	-0.40	-0.26	0.28	0.35
20200	0.60	0.25	0.11	-0.09
19860	0.99	0.00	0.00	0.00
19600	0.01	0.00	0.00	0.00
19200	0.00	0.00	0.00	0.00
18600	0.00	0.00	0.00	0.00
18300	-5.83	-2.86	-6.58	-3.84
17700	-2.52	-1.23	-3.19	-2.26
17100	-0.37	-0.41	-0.27	-0.73

(Continued)

Table 2 (Concluded)

Channel Station No.	Peak of 100 Year Flood		End of 100 Year Flood	
	Without Project	With Project	Without Project	With Project
16700	1.95	1.03	1.59	0.24
16300	2.18	1.12	1.72	0.46
16001	2.61	-0.54	1.83	1.19
15500	-0.26	1.36	2.03	1.76
14900	1.90	-0.51	0.69	0.55
14400	0.57	0.97	0.79	0.79
14020	0.00	0.00	0.00	0.00
13740	-2.20	-0.59	-1.55	-0.06
13440	-2.66	-0.76	-0.56	0.44
13000	1.90	0.11	-0.24	0.33
12600	-1.14	-1.12	-1.05	-0.14
12100	0.78	0.47	0.38	0.37
11601	0.00	0.24	-0.07	0.51
10801	-0.91	-0.23	-1.02	-0.20
10200	-1.59	-0.56	-0.41	0.29
9840	0.00	0.00	0.00	0.00
9400	-1.58	-1.21	-1.58	-1.43
9000	-2.61	-1.00	-1.77	-0.88
8700	0.00	0.00	0.00	0.00
8400	0.00	0.00	0.00	0.00
7600	-1.36	-0.49	-1.35	-0.20
7000	1.97	0.33	1.25	0.35
6400	1.75	1.08	2.26	1.69
5800	2.24	1.22	2.28	2.16
5200	0.59	0.47	1.01	1.37
4660	0.00	0.00	0.01	0.00
4400	-2.14	-1.00	-0.20	-1.83
4100	-0.55	-2.39	0.37	-1.77
3550	2.53	0.39	3.28	0.32
2800	4.35	1.20	1.05	0.57
2100	-2.89	-0.46	0.20	-0.05
1400	0.59	-0.04	0.33	-0.12
1000	-0.43	-0.67	0.10	-0.35
500	2.77	1.19	0.90	0.08
100	-1.81	-0.76	0.01	-0.20
0	-2.52	-0.52	0.35	0.23

Table 3

Damage Potential Assessment - Tropicana Wash

Total Length of Reach (ft)	TROPICANA WASH											
	Left Bank						Right Bank					
	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank Instability	Potential for Scour or Deposition	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank Instability	Potential for Scour or Deposition
Reach 1 Koval Lane to Harmon Avenue												
1500	30	250	Apartments	Low	High	High	80	175	Apartments	Low	High	High
	50	Vacant		Low	High	High	20	100	Apartments	Medium	Medium	High
	20	100	Apartments	Medium	Medium	High						
Reach 2 Harmon Avenue to Hard Rock Cafe Culvert												
1400	60	160	Apartments	Low	Medium	Low	60	100-400	Apartments	Low	Medium	Low
	40	40	Apartments	High	Medium	Low	40	40	Apartments	High	Medium	Low
Reach 3 Concrete Lined Culvert under Hotel and Apartments - 1800 feet												
Reach 4 Culvert outlet to Swenson Street												
1100	60	Vacant		Low	High	Low	60	40	Commercial	High	High	Low
	40	50	Commercial	High	Medium	Low	40	40	Commercial	High	Medium	Low
Reach 5 Swenson Street to Flamingo Road												
600	100	Vacant		Low	High	Low	100	Vacant		Low	High	Low
Reach 6 Flamingo Road to Confluence with Flamingo Wash												
800	100	Vacant		Low	High	High	100	60	Utility	Medium	High	High

Table 4
Damage Potential Assessment - Flamingo Wash

	FLAMINGO WASH												
	Left Bank						Right Bank						
Total Length of Reach (ft)	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank instability	Potential for Scour or Deposition	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank Instability	Potential for Scour or Deposition
Reach 6 Swenson Street to Cambridge Street												
1300	100	250	Apartments	Low	High	High	100	Vacant		Low	High	High
Reach 7 Cambridge Street to Maryland Parkway												
1300	75	Vacant		Low	High	High	40	80	Commercial	Medium	High	High
	25	20	Commercial	High	Medium	High	60	40	Commercial	High	Medium	High
Reach 8 Maryland Parkway to Spencer Street												
3000	25	70	Commercial	Medium	High	High	25	30	Commercial	High	High	High
	25	60	Apartments	Medium	High	High	25	30	Apartments	High	High	High
	25	20	Apartments	High	Low	Low	25	20	Apartments	High	Low	Low
	25	60	Apartments	Medium	High	High	25	60	Apartments	Medium	High	High
Reach 9 Spencer Street to Eastern Avenue - Golf Course												
3000	5	60	Residential	Medium	High	High	5	80	Residential	Medium	High	High
	95	250	Residential	Low	Low	High	95	250	Residential	Low	Low	High
Reach 10 Eastern Avenue to McLeod Drive												
3100	10	80	Commercial	Medium	High	High	10	30	Commercial	High	High	High
	10	Vacant		Low	High	High	30	Vacant		Low	High	High
	10	50	Residential	Medium	High	High	25	50 - 100	Residential	Medium	High	High
	35	Vacant		Low	High	High	35	Vacant		Low	High	High

(Sheet 1 of 3)

Table 4 (Continued)

	FLAMINGO WASH											
	Left Bank						Right Bank					
Total Length of Reach (ft)	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank instability	Potential for Scour or Deposition	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank Instability	Potential for Scour or Deposition
Reach 10 Eastern Avenue to McLeod Drive (Continued)												
	45	100	Synagogue	Medium	High	High						
Reach 11 McLeod Drive to Desert Inn Road												
1400	100	50	Commercial	High	Medium	Low	100	50	Apartments	High	Medium	Low
Reach 12 Desert Inn Road to Mojave Road												
1300	25	40	Commercial	High	Low	Low	25	10	Commercial	High	Low	Low
	75	50 - 100	Apartments	Medium	Low	Low	75	Vacant		Low	Low	Low
Reach 13 Mojave Road to Vegas Valley Drive												
2400	1000	40	Residential	High	High	High	100	40	Residential	High	High	High
Reach 14 Vegas Valley Drive to Boulder Highway												
2200	100	10 - 50	Trailers	High	High	High	100	10 - 50	Trailers	High	High	High
Reach 15 Boulder Highway to Interstate 515												
1650	100	60 - 150	Few Homes	Medium	High	High	100	Vacant		Low	High	High
Reach 16 Interstate 515 to Lamb Boulevard												
2500	100	50	Commercial	High	High	Medium	100	50	Trailers	High	High	Medium
Reach 17 Lamb Boulevard to Nellis Boulevard												
5100	25	50 - 100	Commercial	Medium	High	High	25	60	Commercial	Medium	High	High

(Sheet 2 of 3)

Table 4 (Concluded)

	FLAMINGO WASH											
	Left Bank						Right Bank					
Total Length of Reach (ft)	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank instability	Potential for Scour or Deposition	% of Reach	Distance to Structures (ft)	Type of Structures	Proximity to Structures	Bank Instability	Potential for Scour or Deposition
Reach 17 Lamb Boulevard to Nellis Boulevard (Continued)												
	25	Vacant		Low	High	Medium	25	Vacant		Low	High	Medium
	40	50	Apartments	Medium	High	High	40	40	Apartments	High	High	High
	10	Vacant		Low	High	High	10	20	Commercial	High	High	High
Reach 18 Nellis Boulevard to confluence with Las Vegas Wash - Golf Course												
4500	100	175 - 200	Residential	Low	Low	High	100	125 - 300	Residential	Low	Low	High
	LAS VEGAS WASH											
Reach 18 Flamingo Wash confluence to Concrete Channel												
2000	100	Vacant		Low	High	High	100	Vacant		Low	High	High
Reach 19 Beginning of Concrete Channel to end of concrete Channel												
2300	100	Vacant		Low	Low	Low	100	Vacant		Low	Low	Low
Reach 20 Downstream from Concrete Channel												
900	100	Vacant		Low	Medium	Low	100	350	Utility	Low	Medium	Low

(Sheet 3 of 3)

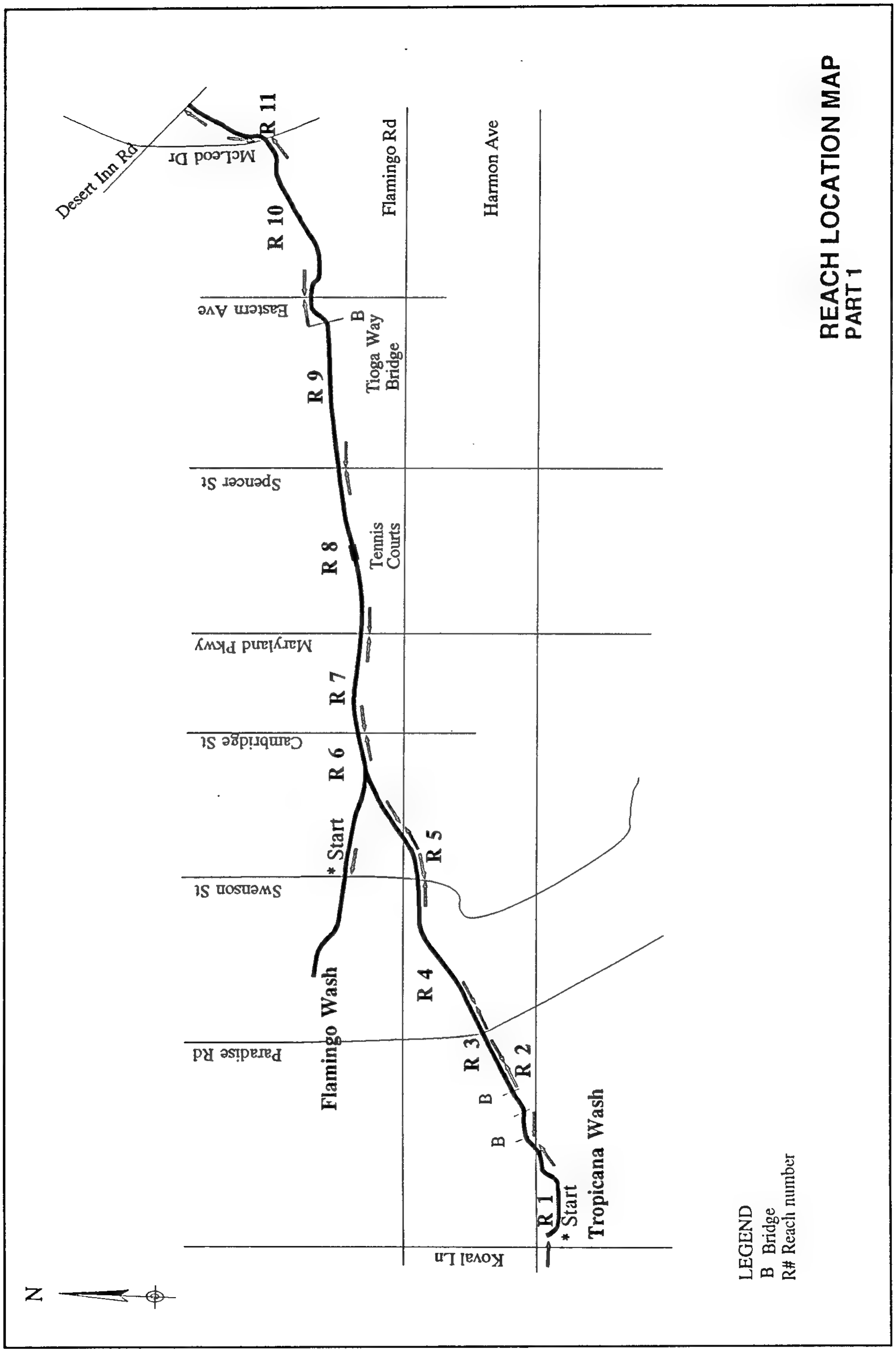

REACH LOCATION MAP
PART 1
Desert Inn Rd
McLeod Dr
Flamingo Rd
Harmon Ave
Eastern Ave
Tioga Way Bridge
Spencer St
Tennis Courts
Maryland Pkwy
Cambridge St
Swenson St
Paradise Rd
Koval Ln
Flamingo Wash
Tropicana Wash
* Start
* Start
R 1
R 2
R 3
R 4
R 5
R 6
R 7
R 8
R 9
R 10
R 11
B
B
B
N
LEGEND
B Bridge
R# Reach number

Plate 1

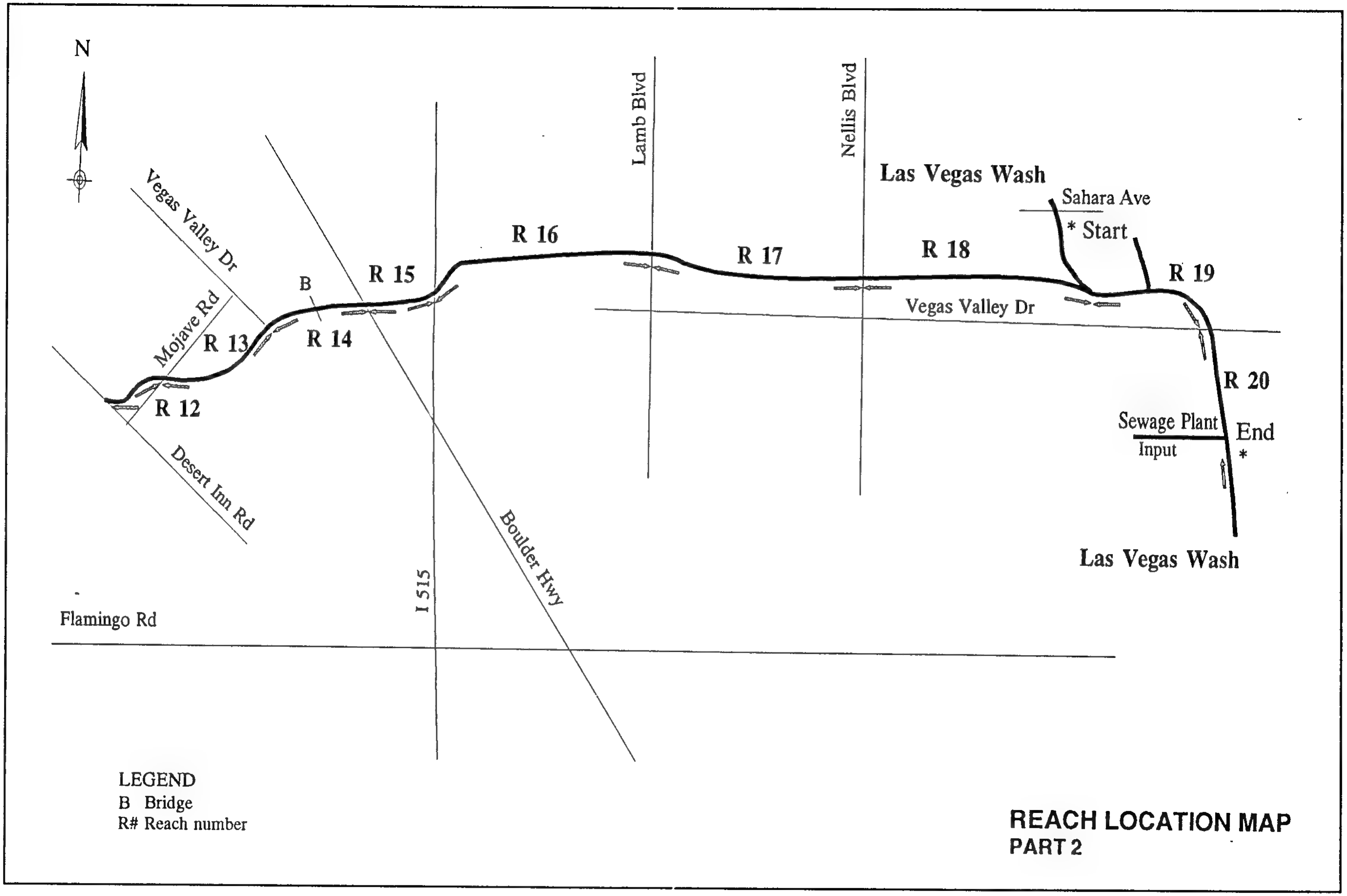
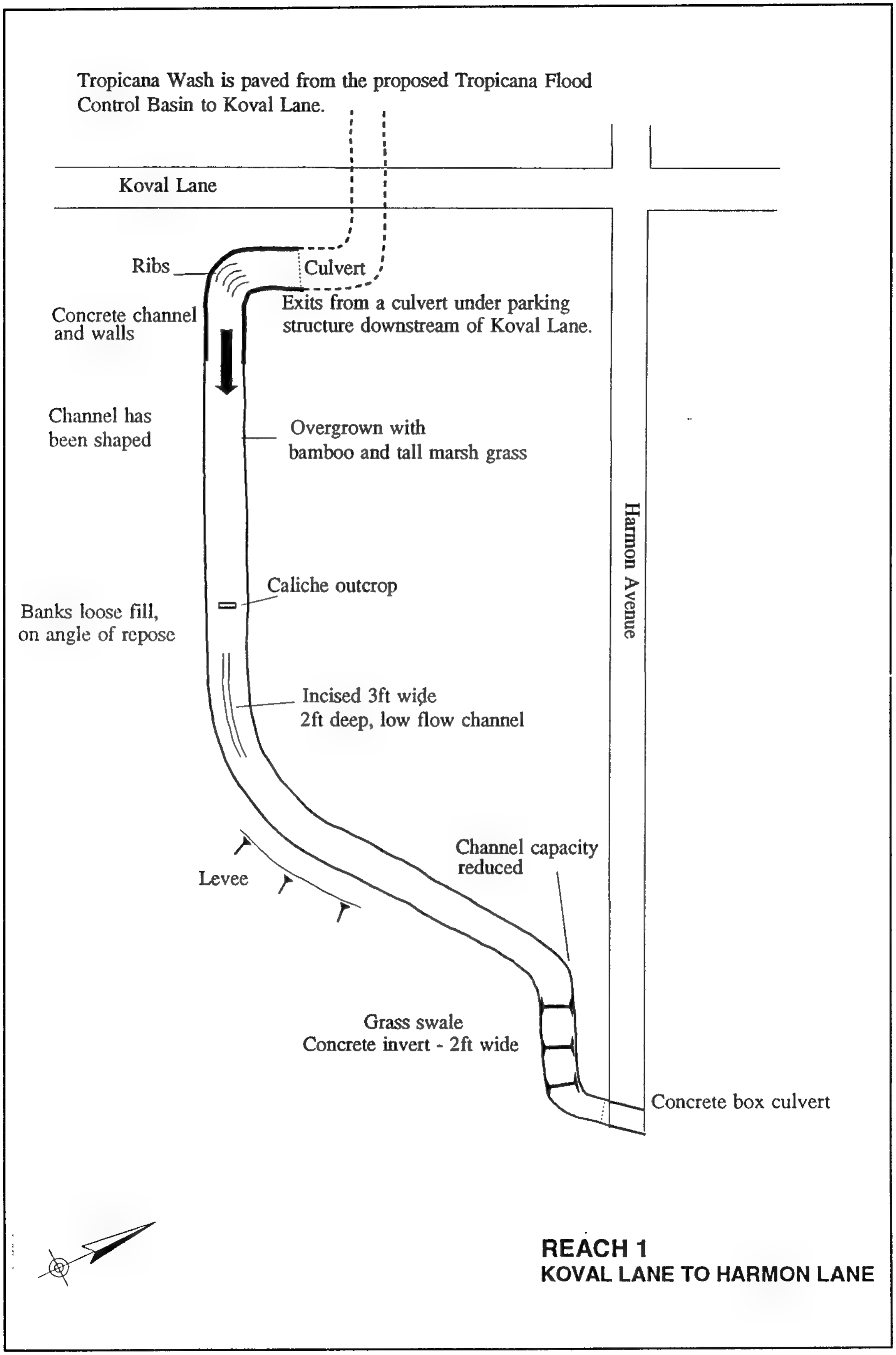
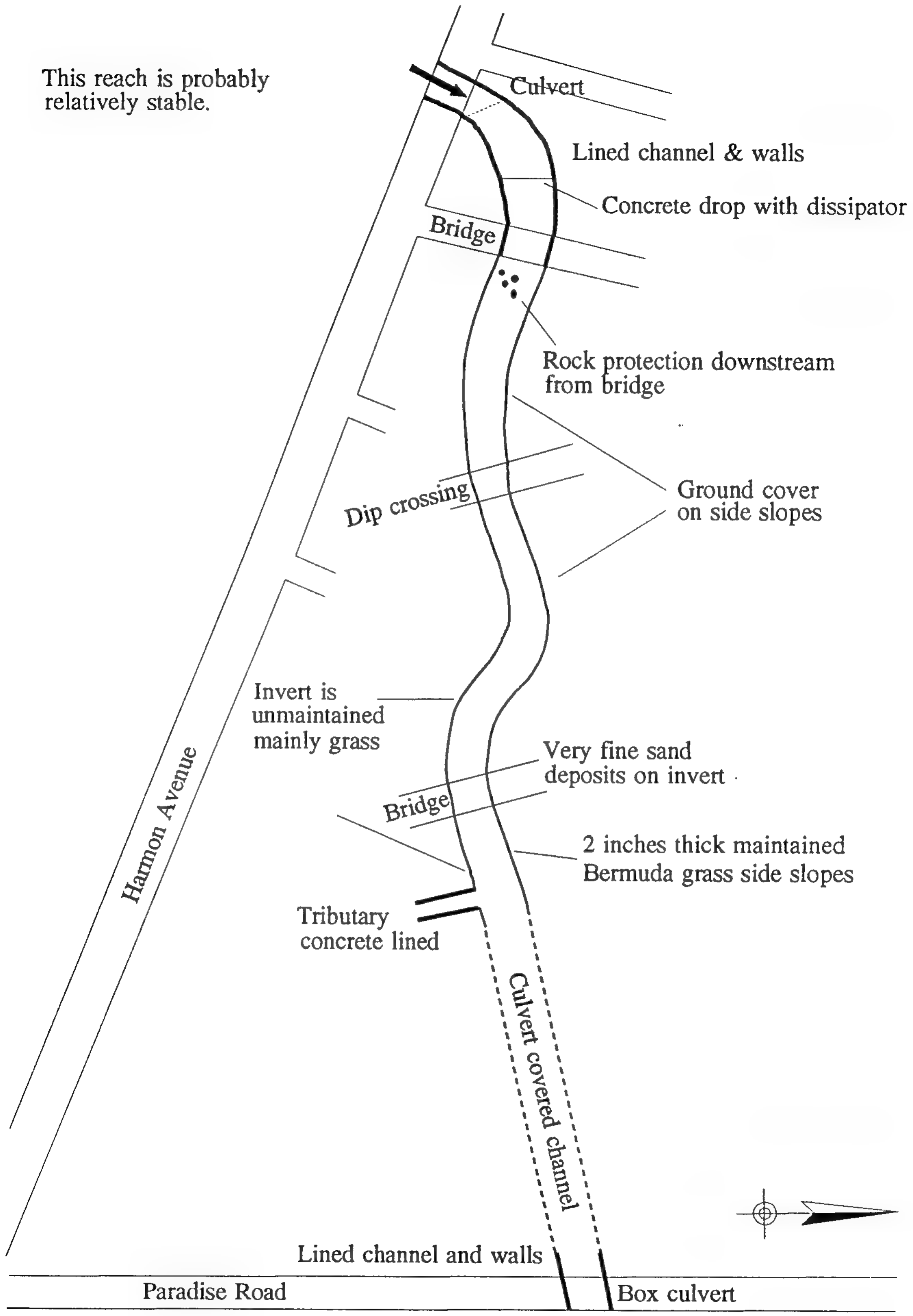

N
Vegas Valley Dr
Mojave Rd
Desert Inn Rd
Flamingo Rd
I 515
Boulder Hwy
Lamb Blvd
Nellis Blvd
Las Vegas Wash
Sahara Ave
* Start
R 12
R 13
B
R 14
R 15
R 16
R 17
R 18
R 19
Vegas Valley Dr
R 20
Sewage Plant
Input
End
*
Las Vegas Wash
LEGEND
B Bridge
R# Reach number
REACH LOCATION MAP
PART 2

Plate 2

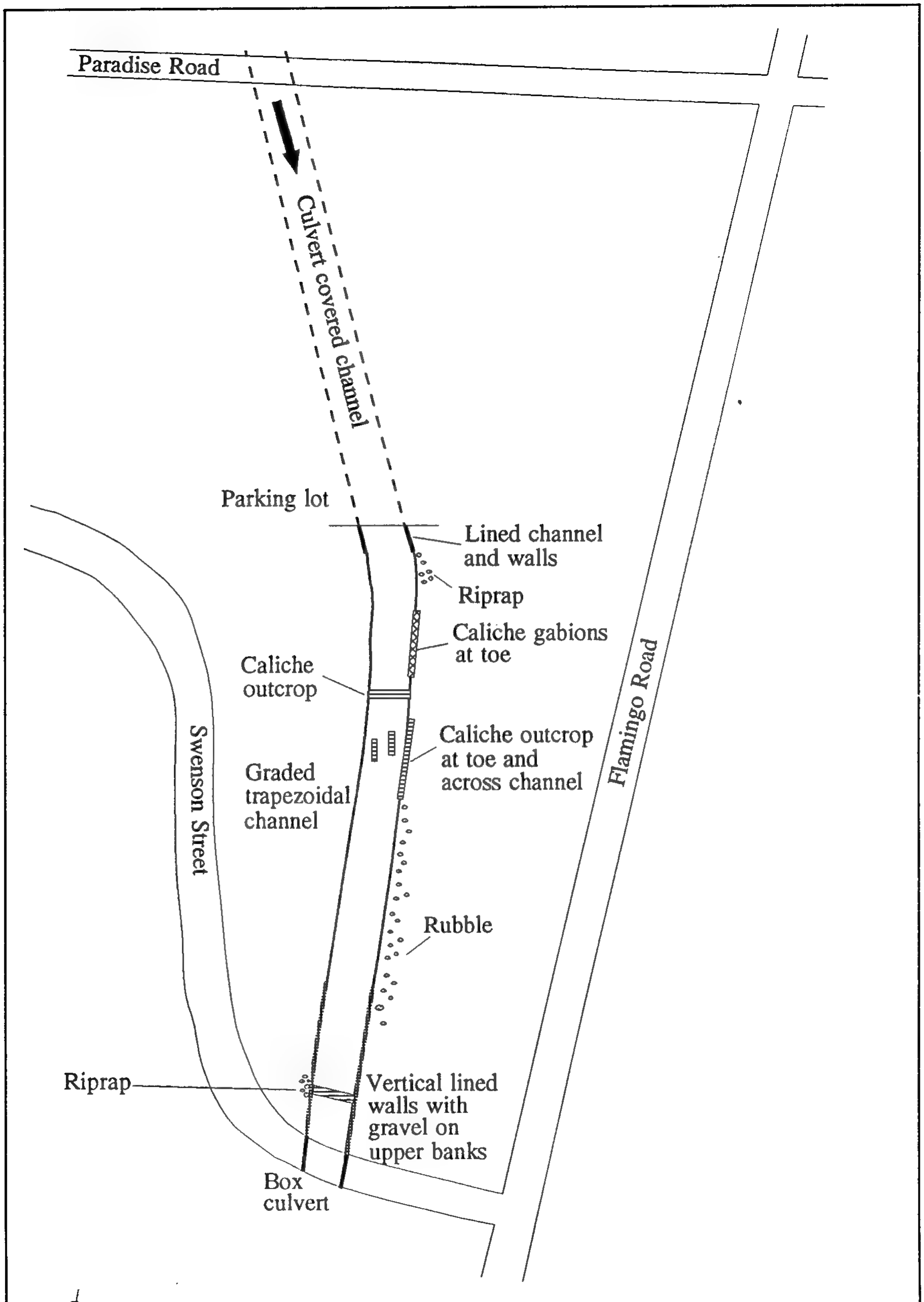


Tropicana Wash is paved from the proposed Tropicana Flood Control Basin to Koval Lane.
Koval Lane
Ribs
Culvert
Concrete channel and walls
Exits from a culvert under parking structure downstream of Koval Lane.
Channel has been shaped
Overgrown with bamboo and tall marsh grass
Harmon Avenue
Caliche outcrop
Banks loose fill, on angle of repose
Incised 3ft wide 2ft deep, low flow channel
Channel capacity reduced
Levee
Grass swale Concrete invert - 2ft wide
Concrete box culvert
REACH 1
KOVAL LANE TO HARMON LANE



REACH 2
HARMON AVENUE TO PARADISE ROAD

Plate 4



REACH 4
PARADISE ROAD TO SWENSON STREET

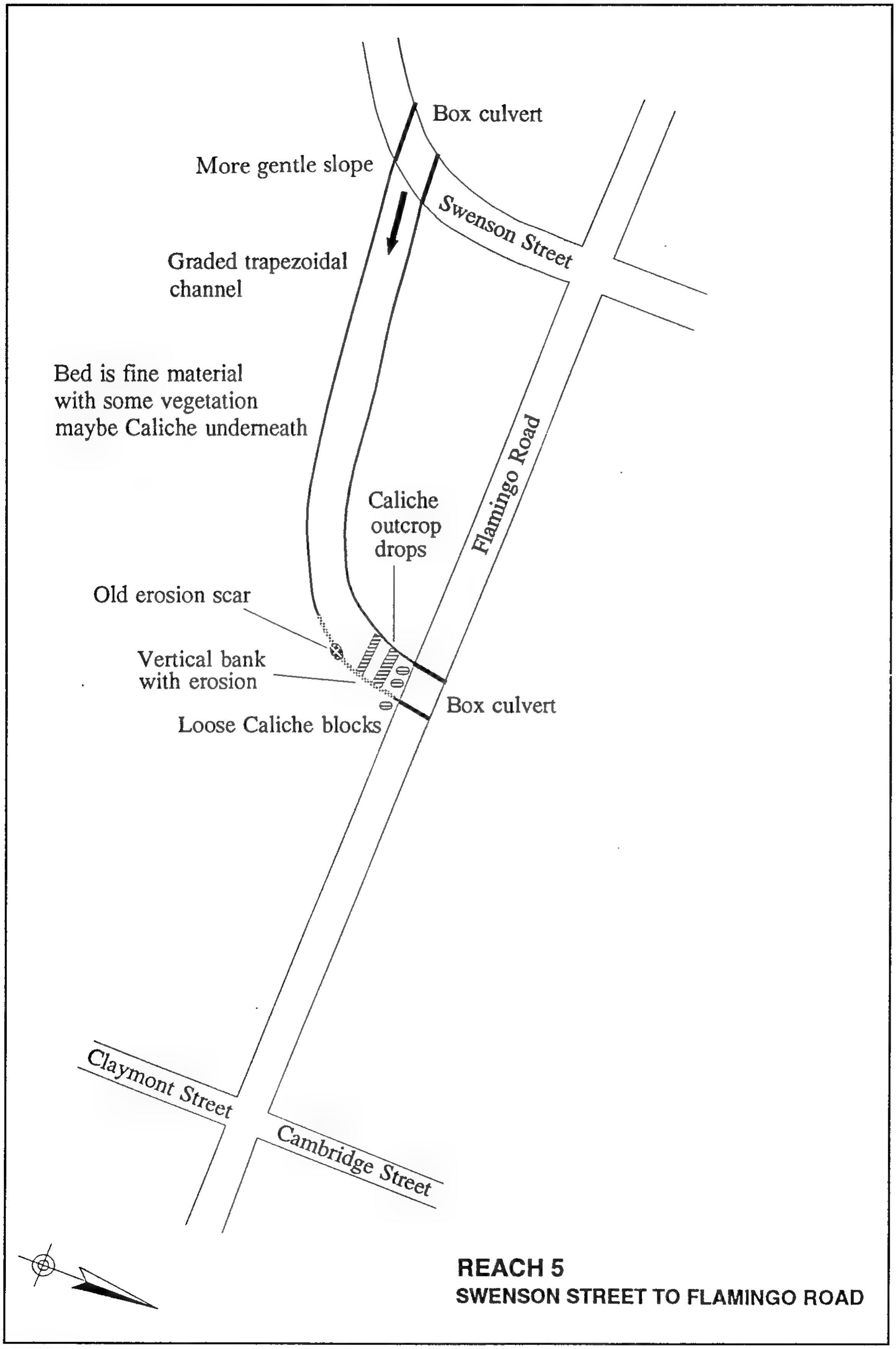
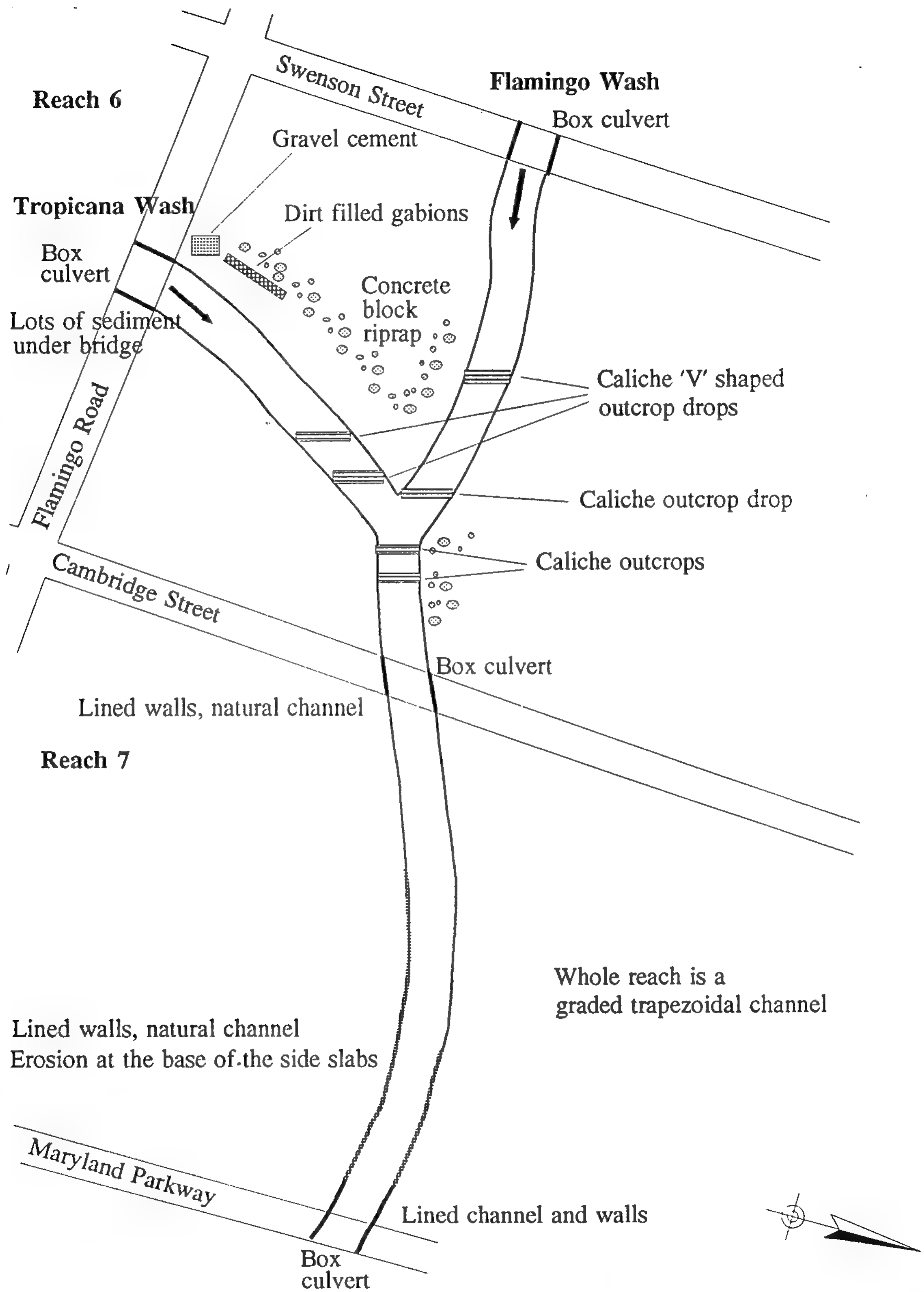
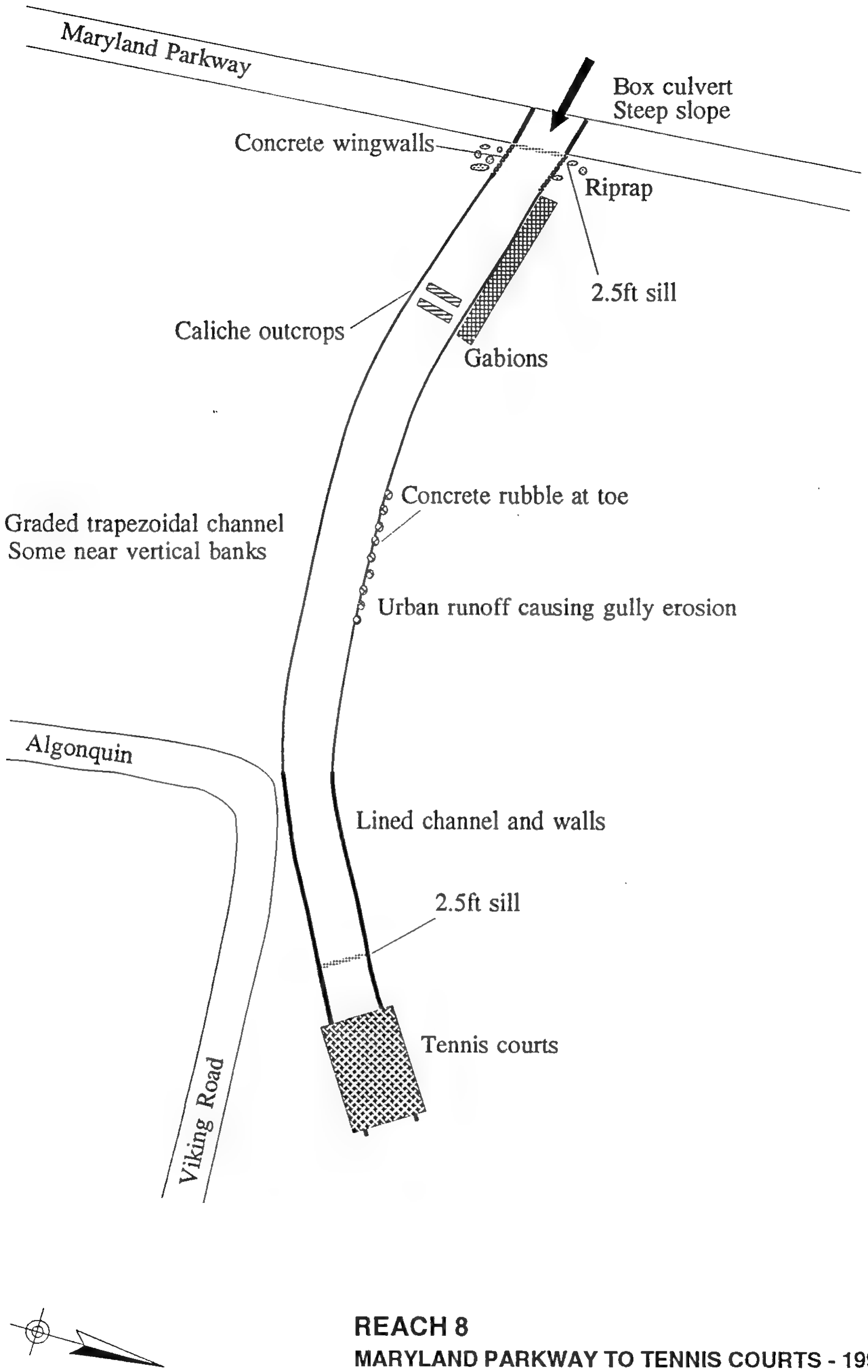

Box culvert
More gentle slope
Swenson Street
Graded trapezoidal
channel
Bed is fine material
with some vegetation
maybe Caliche underneath
Flamingo Road
Caliche
outcrop
drops
Old erosion scar
Vertical bank
with erosion
Box culvert
Loose Caliche blocks
Claymont Street
Cambridge Street
REACH 5
SWENSON STREET TO FLAMINGO ROAD

Plate 6



REACHES 6 & 7

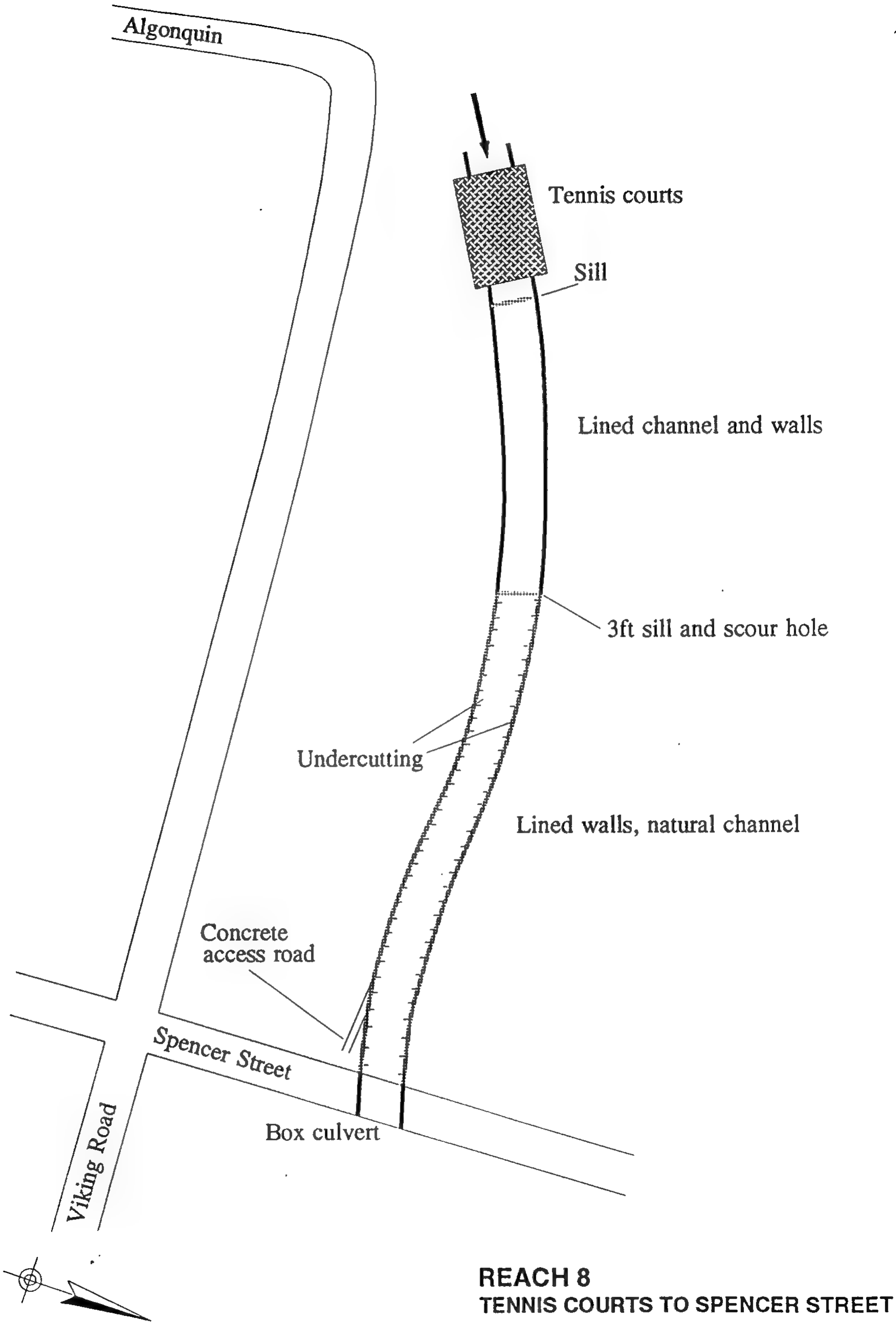
REACH 6 CONFLUENCE OF TROPICANA AND FLAMINGO WASHES AT CAMBRIDE STREET
REACH 7 FLAMINGO WASH FROM CAMBRIDGE STREET TO MARYLAND PARKWAY



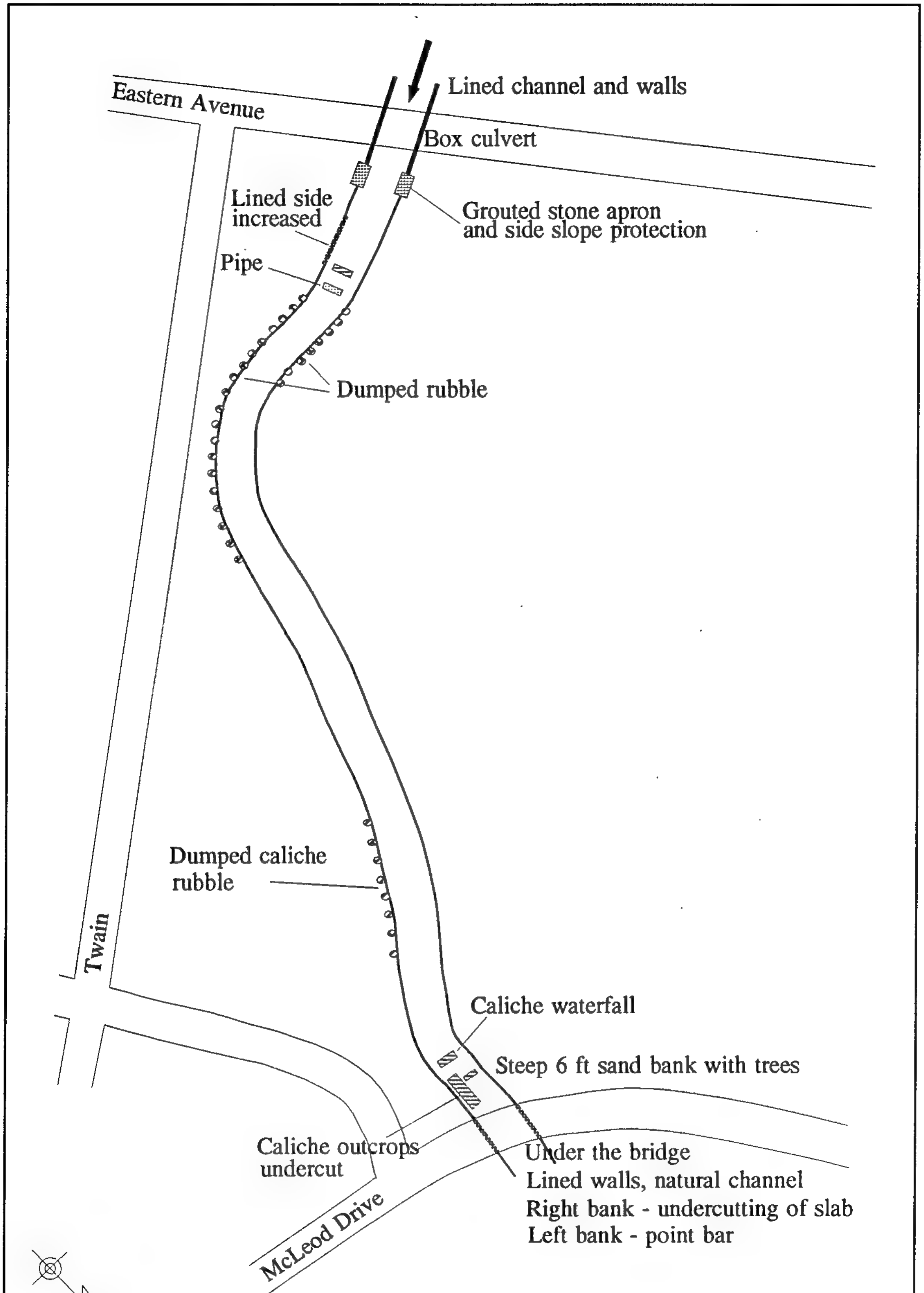
REACH 8

MARYLAND PARKWAY TO TENNIS COURTS - 1995

Plate 8

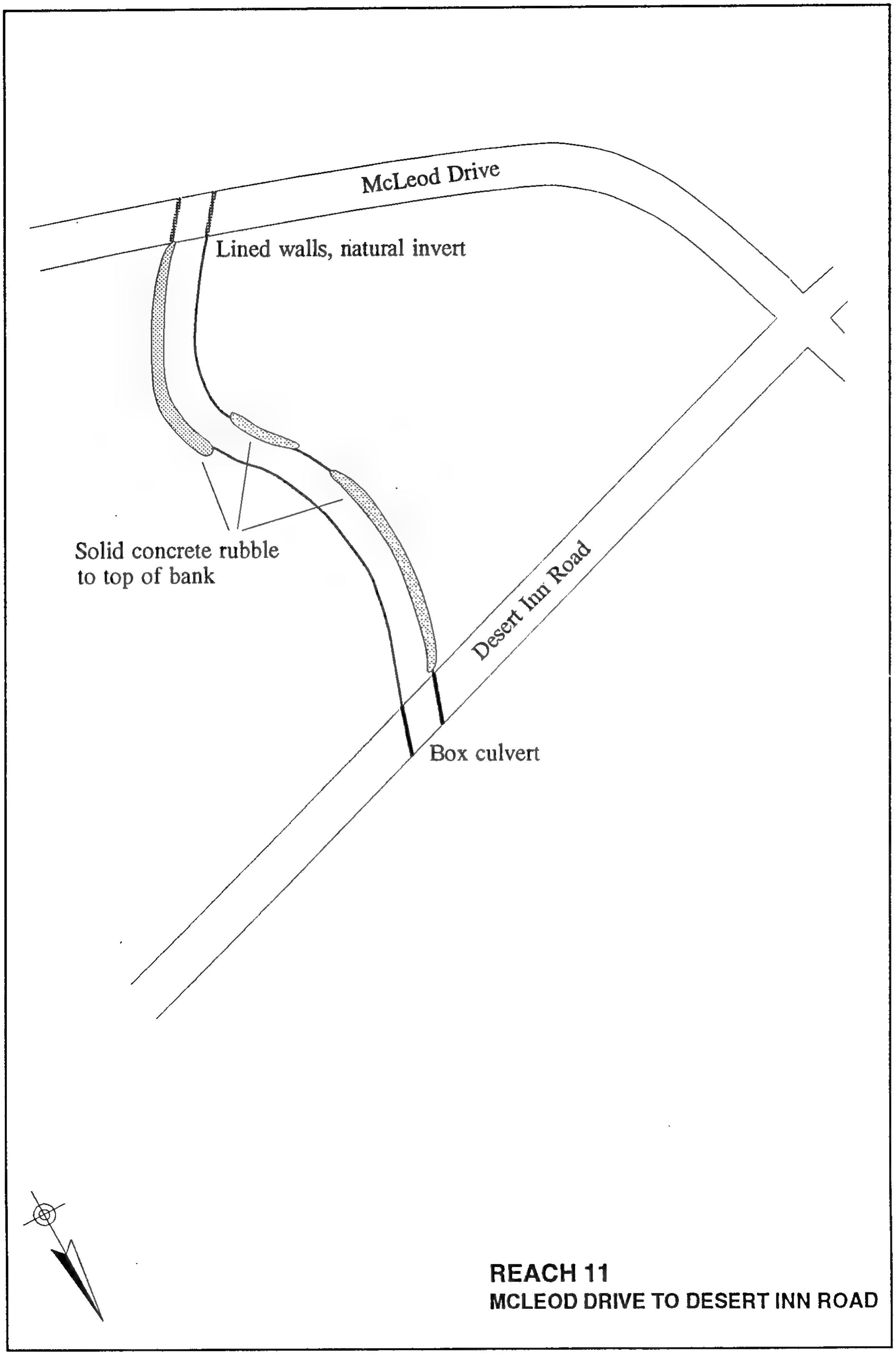


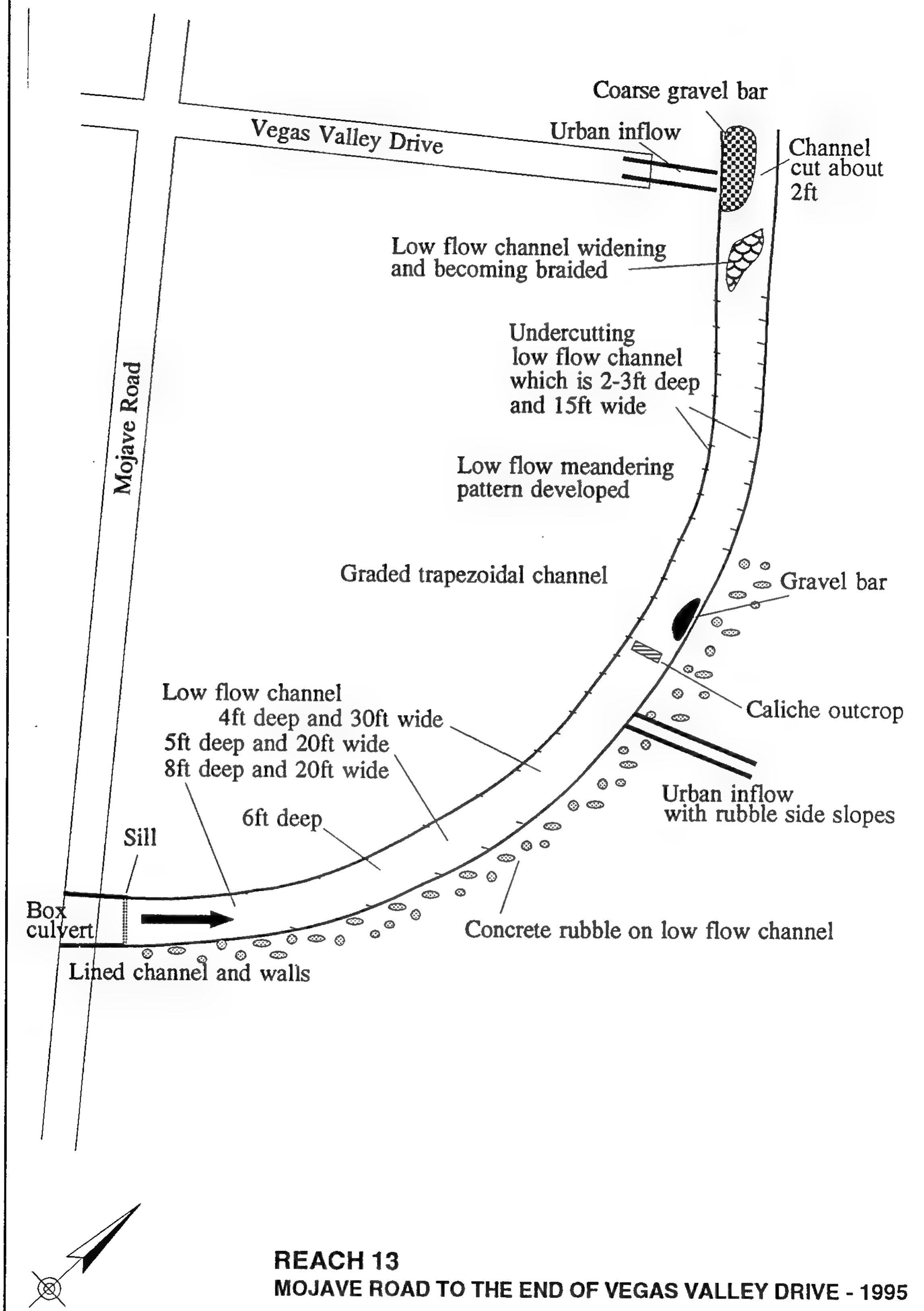
REACH 8
TENNIS COURTS TO SPENCER STREET



REACH 10
EASTERN AVENUE TO MCLEOD DRIVE - 1995

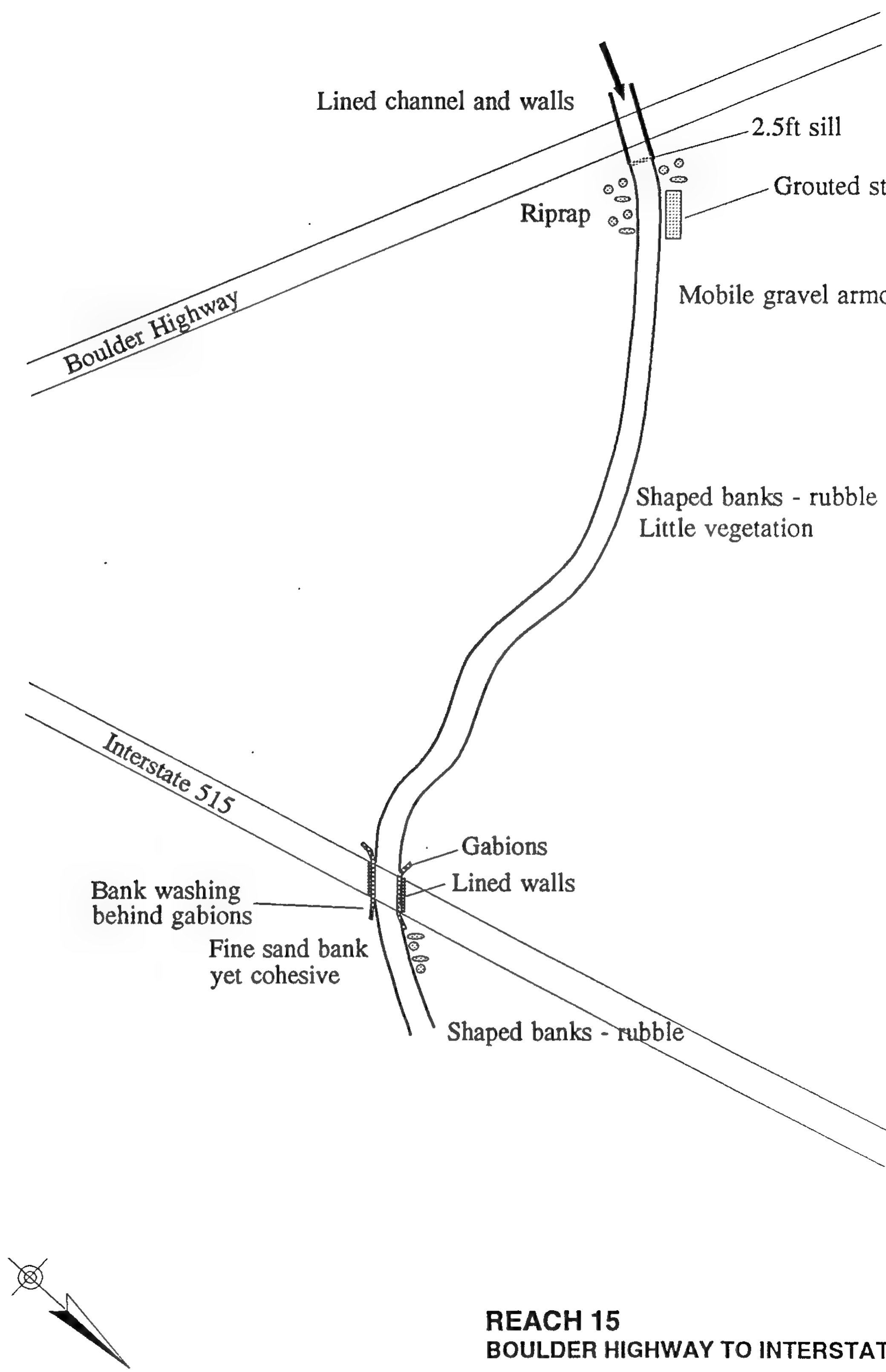
Plate 10


McLeod Drive
Lined walls, natural invert
Solid concrete rubble
to top of bank
Desert Inn Road
Box culvert
REACH 11
MCLEOD DRIVE TO DESERT INN ROAD



REACH 13
MOJAVE ROAD TO THE END OF VEGAS VALLEY DRIVE - 1995

Plate 12



REACH 15
BOULDER HIGHWAY TO INTERSTATE 515

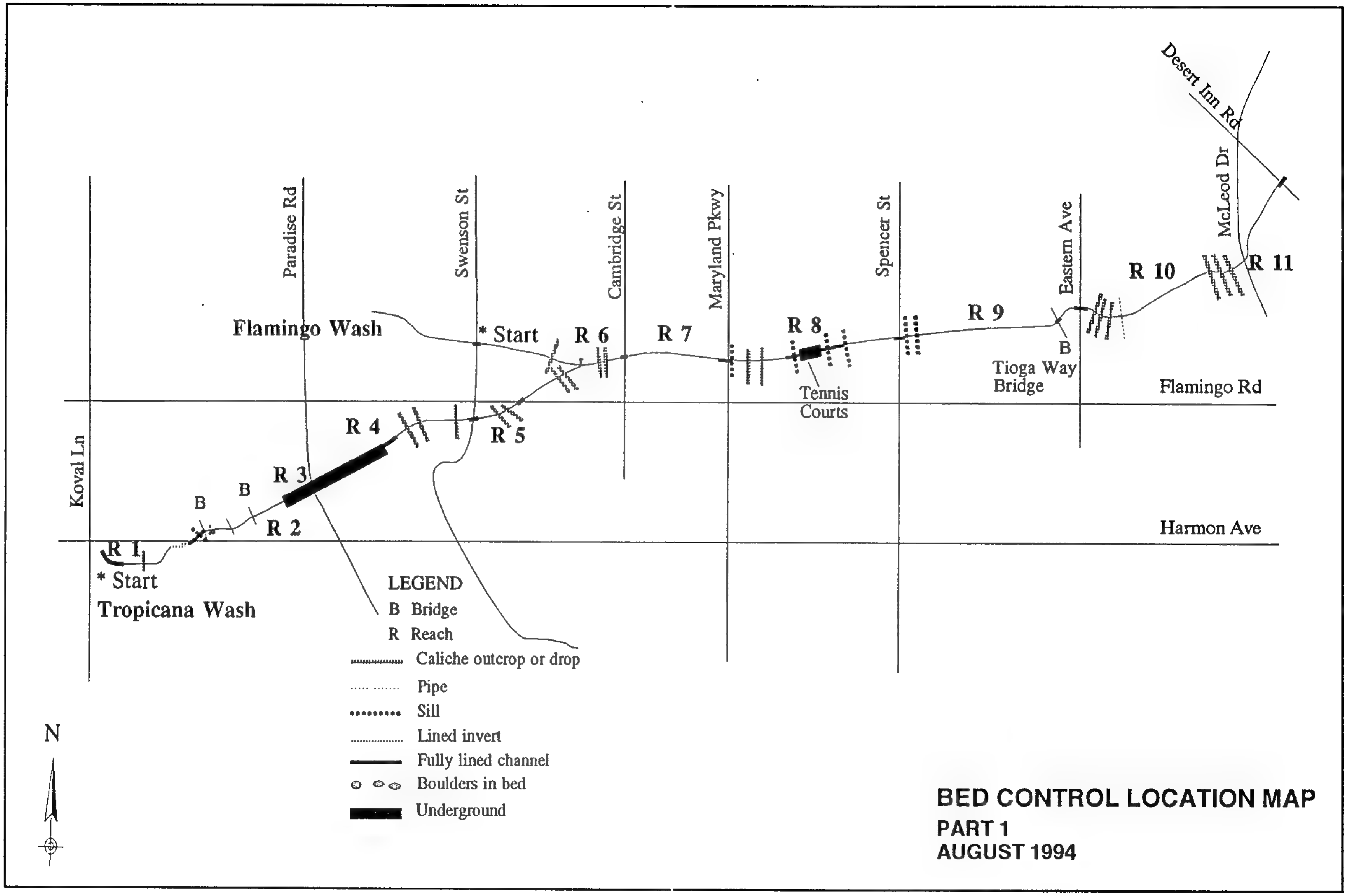
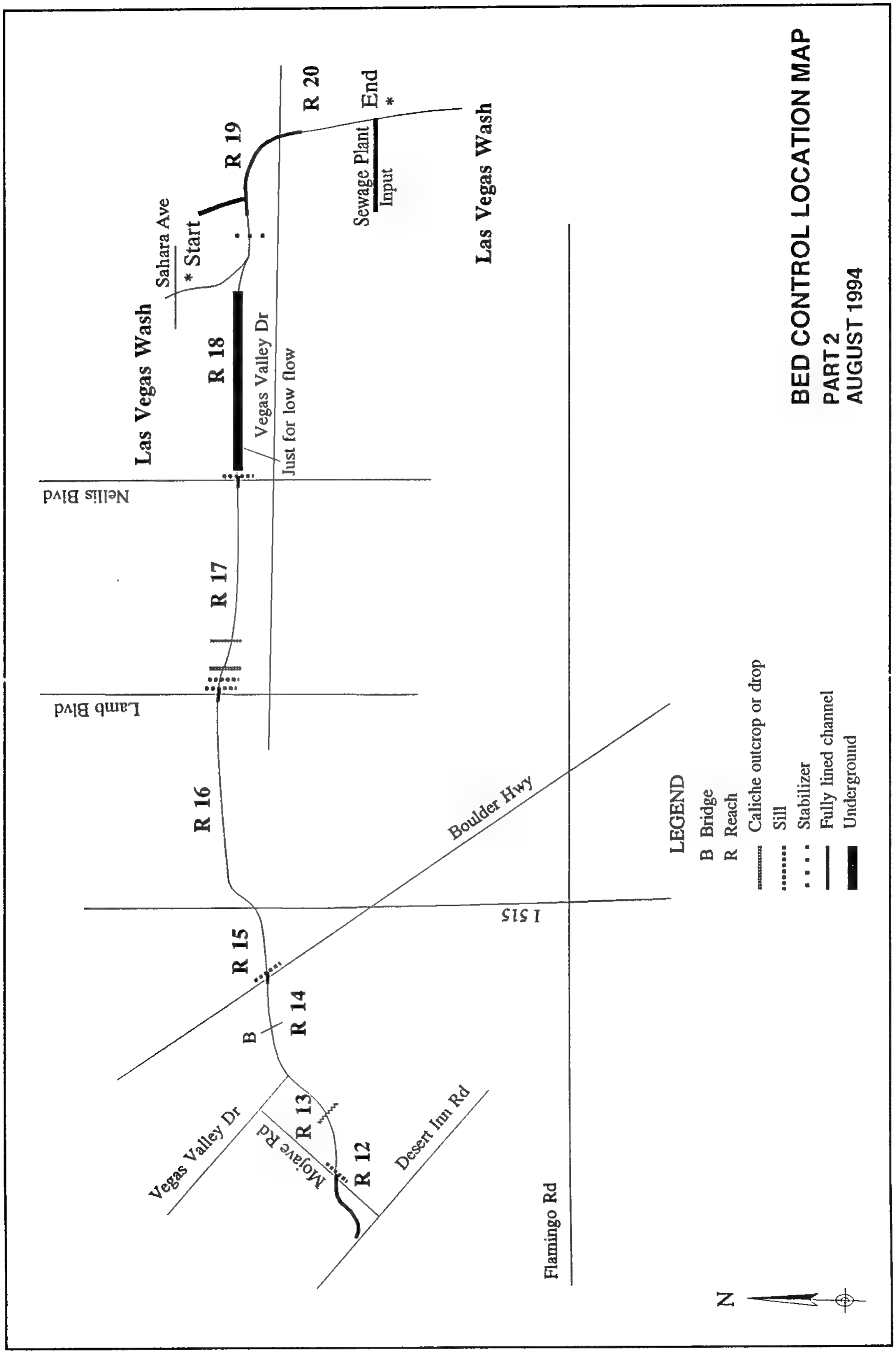

BED CONTROL LOCATION MAP
PART 1
AUGUST 1994
LEGEND
B Bridge
R Reach
Caliche outcrop or drop
Pipe
Sill
Lined invert
Fully lined channel
Boulders in bed
Underground
N
Koval Ln
Paradise Rd
Swenson St
Cambridge St
Maryland Pkwy
Spencer St
Eastern Ave
McLeod Dr
Desert Inn Rd
Flamingo Rd
Harmon Ave
Flamingo Wash
Tropicana Wash
* Start
* Start
R 1
R 2
R 3
R 4
R 5
R 6
R 7
R 8
R 9
R 10
R 11
B
B
B
Tioga Way Bridge
Tennis Courts

Plate 14



BED CONTROL LOCATION MAP
PART 2
AUGUST 1994
Las Vegas Wash
Sahara Ave
* Start
R 18
Vegas Valley Dr
Just for low flow
R 19
R 20
End
*
Sewage Plant
Input
Las Vegas Wash
Nellis Blvd
R 17
Lamb Blvd
R 16
Boulder Hwy
LEGEND
B Bridge
R Reach
Caliche outcrop or drop
Sill
Stabilizer
Fully lined channel
Underground
I 515
R 15
R 14
B
R 13
R 12
Vegas Valley Dr
Mojave Rd
Desert Inn Rd
Flamingo Rd
N

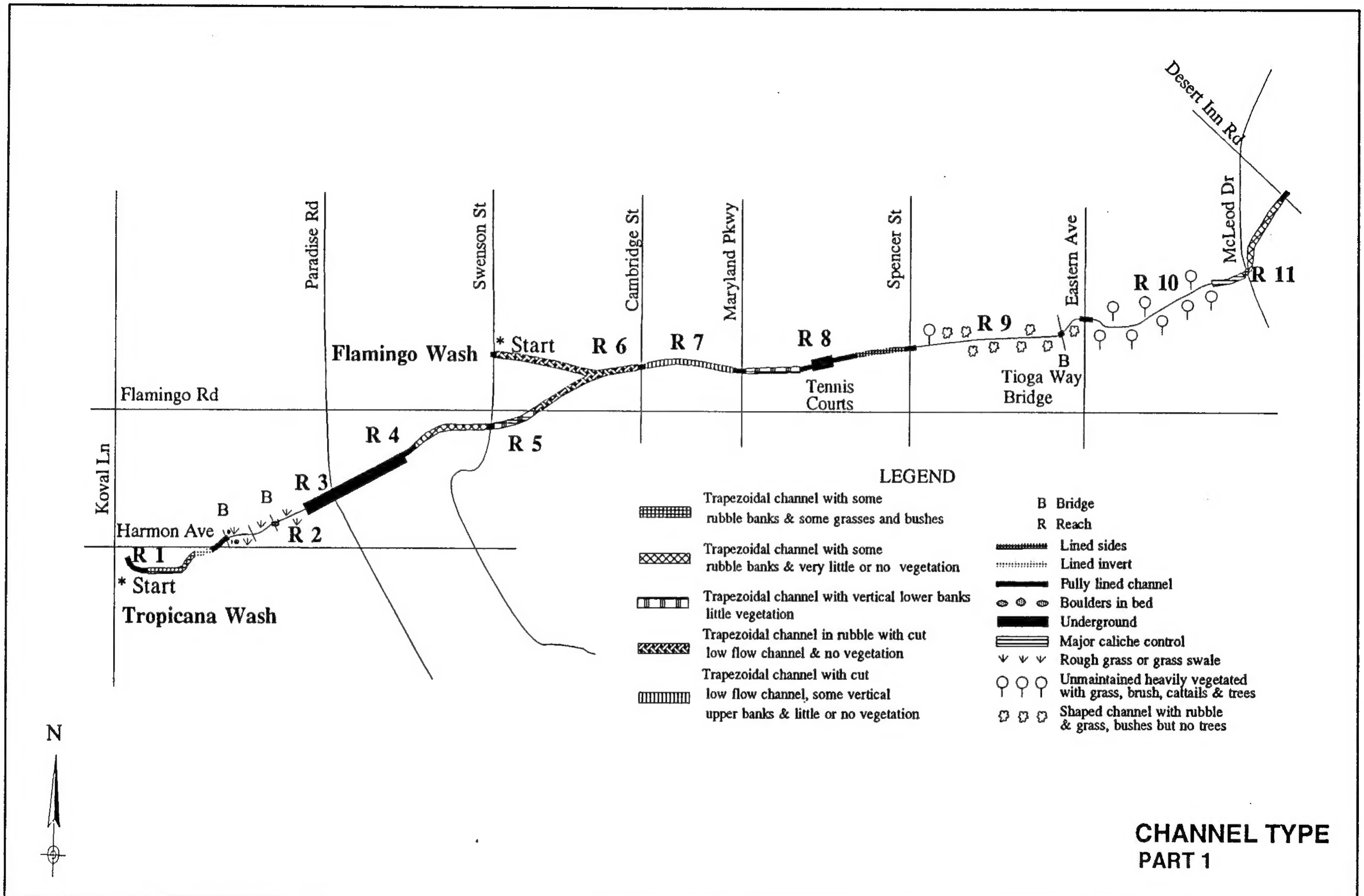
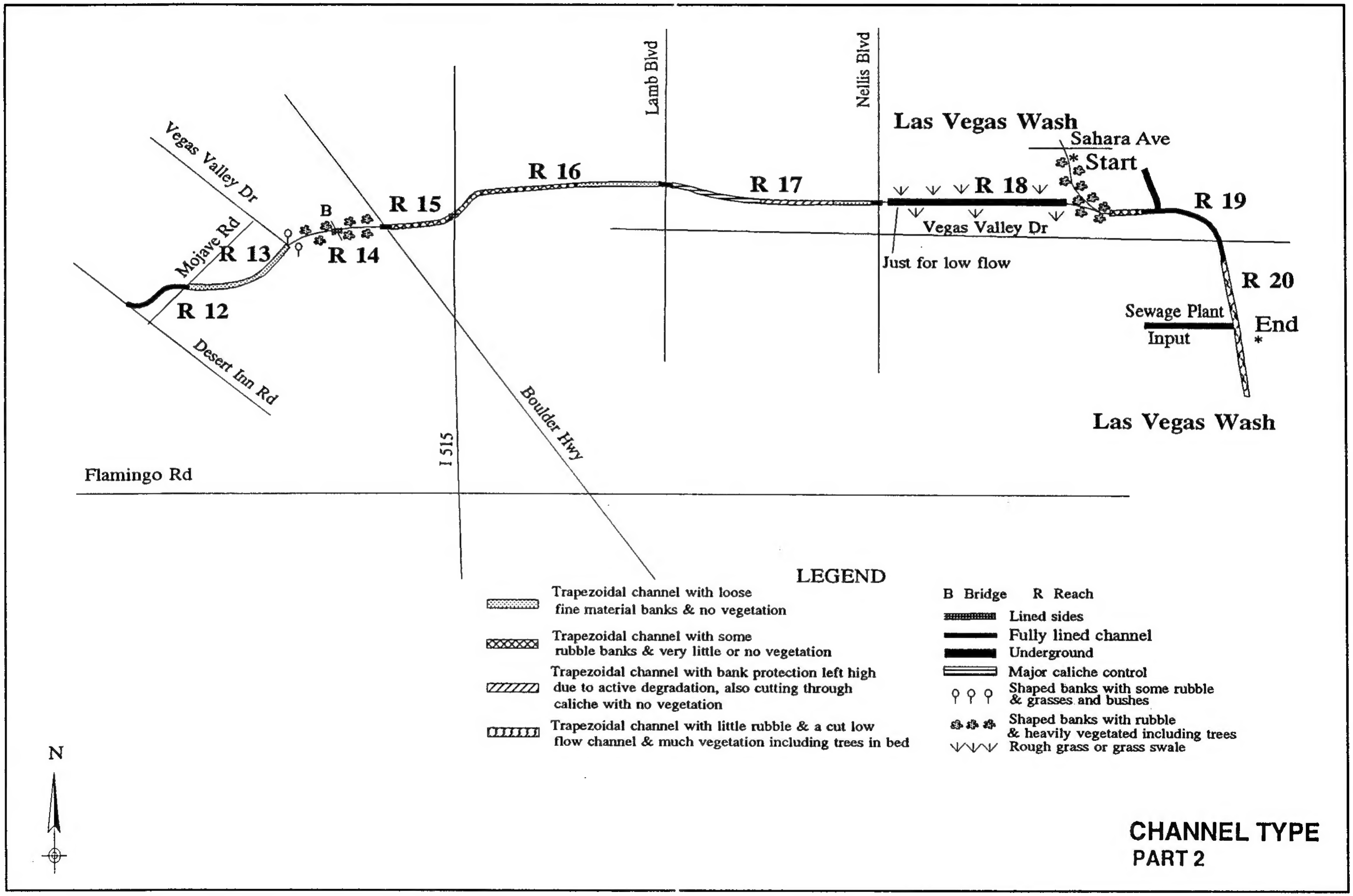

Flamingo Wash
* Start
Tropicana Wash
* Start
Koval Ln
Flamingo Rd
Harmon Ave
Paradise Rd
Swenson St
Cambridge St
Maryland Pkwy
Spencer St
Eastern Ave
McLeod Dr
Desert Inn Rd
Tennis Courts
Tioga Way Bridge
R 1
R 2
R 3
R 4
R 5
R 6
R 7
R 8
R 9
R 10
R 11
B
B
B
N
LEGEND
Trapezoidal channel with some rubble banks & some grasses and bushes
Trapezoidal channel with some rubble banks & very little or no vegetation
Trapezoidal channel with vertical lower banks little vegetation
Trapezoidal channel in rubble with cut low flow channel & no vegetation
Trapezoidal channel with cut low flow channel, some vertical upper banks & little or no vegetation
B Bridge
R Reach
Lined sides
Lined invert
Fully lined channel
Boulders in bed
Underground
Major caliche control
Rough grass or grass swale
Unmaintained heavily vegetated with grass, brush, cattails & trees
Shaped channel with rubble & grass, bushes but no trees
CHANNEL TYPE
PART 1

Plate 16


Las Vegas Wash
Sahara Ave
Start
End
Las Vegas Wash
Vegas Valley Dr
Mojave Rd
Desert Inn Rd
Flamingo Rd
Boulder Hwy
I 515
Lamb Blvd
Nellis Blvd
Vegas Valley Dr
Just for low flow
Sewage Plant
Input
R 12
R 13
R 14
B
R 15
R 16
R 17
R 18
R 19
R 20
N
LEGEND
Trapezoidal channel with loose fine material banks & no vegetation
Trapezoidal channel with some rubble banks & very little or no vegetation
Trapezoidal channel with bank protection left high due to active degradation, also cutting through caliche with no vegetation
Trapezoidal channel with little rubble & a cut low flow channel & much vegetation including trees in bed
B Bridge R Reach
Lined sides
Fully lined channel
Underground
Major caliche control
Shaped banks with some rubble & grasses and bushes
Shaped banks with rubble & heavily vegetated including trees
Rough grass or grass swale
CHANNEL TYPE
PART 2

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY ***(Leave blank)***

2. REPORT DATE
September 1996

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
Channel Stability Study, Flamingo Wash, Las Vegas, Nevada

5. FUNDING NUMBERS

6. AUTHOR(S)
Ronald R. Copeland, Lisa C. Hubbard

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER
Technical Report HL-96-18

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Engineer District, Los Angeles
911 Wilshire Boulevard
Los Angeles, CA 90017

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT

Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT ***(Maximum 200 words)***

The channel stability assessment for Flamingo and Tropicana Washes was conducted to assess the potential for change in channel stability associated with the implementation of a proposed U.S. Army Corps of Engineers flood control plan for the city of Las Vegas. The flood control plan included flood detention basins that would store the peak flood flows and release them at a lower flowrate over a longer period of time. Reaches of Flamingo and Tropicana Washes downstream from the proposed Tropicana detention basin were studied. The reaches of Flamingo and Tropicana Washes studied are atypical of alluvial streams in that the bed is not fully mobile, and there are numerous natural and man-made bed controls. A numerical model simulation was conducted to assess the difference in the vertical stability of the existing channel for the with-project and the without-project one-percent-chance exceedance hydrographs. The HEC-6W one-dimensional numerical sedimentation program was used to develop the numerical model for this study. The results from the model are useful for comparing the potential for vertical channel instability for existing conditions and for post-project conditions.

(Continued)

14. SUBJECT TERMS

Flamingo Wash	Tropocana Wash	Las Vegas, NV
Stable channel	HEC-6W	SAM

15. NUMBER OF PAGES
99

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

13. (Concluded).

A comparison of calculated with- and without-project results on Flamingo Wash demonstrates that degradation and aggradation would be roughly twice as severe for the without-project hydrograph. It can be inferred that decreasing the aggradation and degradation potential will also decrease channel instability. A channel inventory was conducted to identify channel characteristics that would affect channel stability. A damage potential assessment was also made, with estimates based on the distance of structures from the streambank. A channel stability assessment was also made using both the allowable velocity and stable channel methods. These analyses show that although the channel instability problems will be less severe, they will continue to exist with the flood control project in place.